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Surveying and Mapping Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

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BACK ISSUES OF THE DIVISION JOURNALS

----are in such demand that staff headquarters finds itself without a complete record set. Such a set is being assembled for the ASCE Publications Record Room at the United Engineering Center, soon to be dedicated in New York City. The record set of Proceedings reprints is complete; but the Journal file lacks the March, 1956 Journal of the Surveying and Mapping Division.

Members who can spare a copy will be doing a service to the Society by mailing it to Harold T. Larsen, ASCE, Room 1607, 33 West 39th Street, New York 18, New York.

Vol. 87 No. SU 2

Journal of the

SURVEYING AND MAPPING DIVISION

Proceedings of the American Society of Civil Engineers

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Journal of the
SURVEYING AND MAPPING DIVISION
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SURVEYS FOR SPACECRAFT TRACKING SITES

By Robert C. Wigand,[1] M. ASCE

SYNOPSIS

Surveying procedures used in selection of sites for tracking space vehicles involve new applications of celestial surveying and surveying mathematics. Mapping the proposed trajectory is useful in selecting location of stations, particularly for tracking low-altitude satellites. Mapping may be accomplished by establishing an equation in spherical coordinates based on the trajectory criteria. In comparative evaluation of sites, it is desirable to determine the acquisition and departure azimuths to the spacecraft, and to measure horizon clearances so that the effect of obstructions to tracking may be compared. Special requirements also pertain to geodetic and layout surveys of these sites. Further application of surveying techniques is required in the development of spacedraft tracking.

INTRODUCTION

Introduction of the word "spacecraft" to technical vocabulary provides a convenient term to designate vehicles in earth orbit or on a departure trajectory from the earth.[2] This paper describes the principal surveying procedures used in the selection of sites for spacecraft tracking stations, with emphasis

Note.—Discussion open until December, 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Surveying and Mapping Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SU 2, July, 1961.

1 Asst. Field Supervisor, Burns and Roe, Inc., New York, N. Y.

2 Testimony by T. Keith Glennan, Administrator, Natl. Aeronautics and Space Administration before the NASA Authorization Subcommittee of the Senate Committee on Aeronautical and Space Sciences, March 28, 1960.

on applications of celestial surveying and surveying mathematics that are unique to this field.

Surveying is applied to several phases of tracking station site selection. These include 1. mapping of the proposed spacecraft trajectory, 2. determination of acquisition azimuths, 3. measurement of horizon clearances, 4. rough layout of station facilities, and 5. property description.

MAPPING THE PROPOSED TRAJECTORY

One of the first steps after establishment of the trajectory criteria by the National Aeronautics and Space Administration for a manned orbital flight program is to plot the proposed track on a world map. It is necessary in the case of a low-altitude manned satellite, such as the Project Mercury capsule, to select tracking locations from which the spacecraft will be within the 500 mile to 700 mile range of highly directional tracking radars. In the case of deep-space probes, range limitations are not so directly related to geographic location and a wider choice of areas is possible.

The track represents the line of points on the earth's surface over which the spacecraft will make a zenith pass. Practical geography often requires that stations be located slightly off this line in order to take advantage of available land and to be within range on more than one pass.

The compressed time-phasing of a national effort such as Project Mercury requires construction of the tracking station network concurrently with the development of vehicle and communication systems. Consequently, the reliability with which the vehicle will be able to meet the specified orbit becomes a decision-demanding problem to the siting engineer before it becomes one to the Astronaut. The civil engineers who select, lay out and construct tracking stations must give forethought to whatever breadth the criteria specifications provide in considering deviations from the nominal programmed trajectory.

The general areas in which stations are to be located may be selected by map study. Later, when specific sites have been chosen, their precise geocentric locations can be determined by astronomical observations or geodetic tie-ins. Laplace azimuth determinations may also be made after site selection.

Elements of a circular orbit, simplified, are inclination, altitude, period of revolution, and longitude of a node. An elliptical orbit may be designated by an expanded series of elements.[3]

Utilizing an equation that represents the trajectory helps in plotting the track. Fig. 1 shows the periodic curve traced by a proposed Project Mercury trajectory. A prelaunch node is established, constructively, by projecting backward from the insertion point off Cape Canaveral to determine a common reference point in the equatorial plane from which earth rotation and motion of the satellite can be correlated.

The equation for a nominally circular orbit such as the Project Mercury trajectory, is

$$\tan \lambda = \tan \iota \sin (k \theta) \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which λ is latitude, θ is longitude, ι is inclination and, for an easterly launch, $k = 1 + p/23.934$, the term p being the period of one revolution of the spacecraft in hours. It should be noted that θ is measured from the prelaunch node

3 "Observations of Satellite I," by F. L. Whipple and J. A. Hynek, Scientific American, December, 1957.

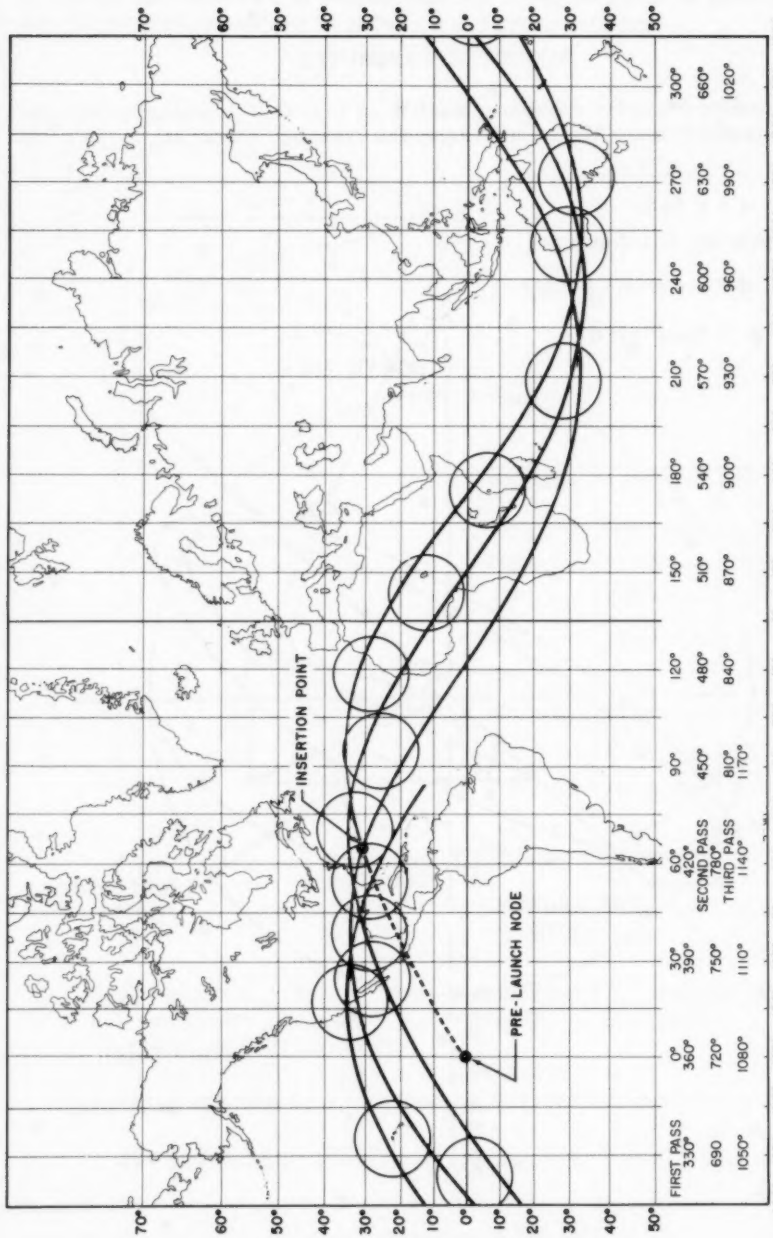


FIG. 1.—TRACK ON WORLD MAP

rather than Greenwich meridian. The cumulative degrees of longitude from the prelaunch node are shown for all three passes at the bottom of Fig. 1.

ACQUISITION AZIMUTHS

In choosing a site for a tracking station, as well as in tracking operations, it is desirable to know the azimuth from the tracking instrument to the point

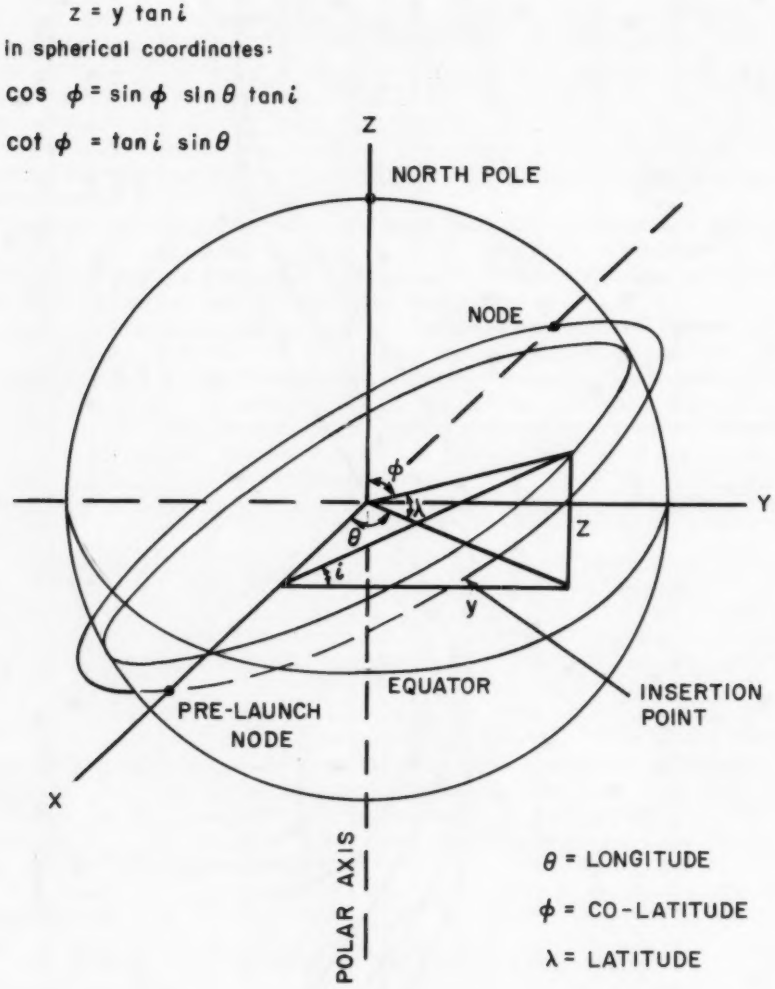
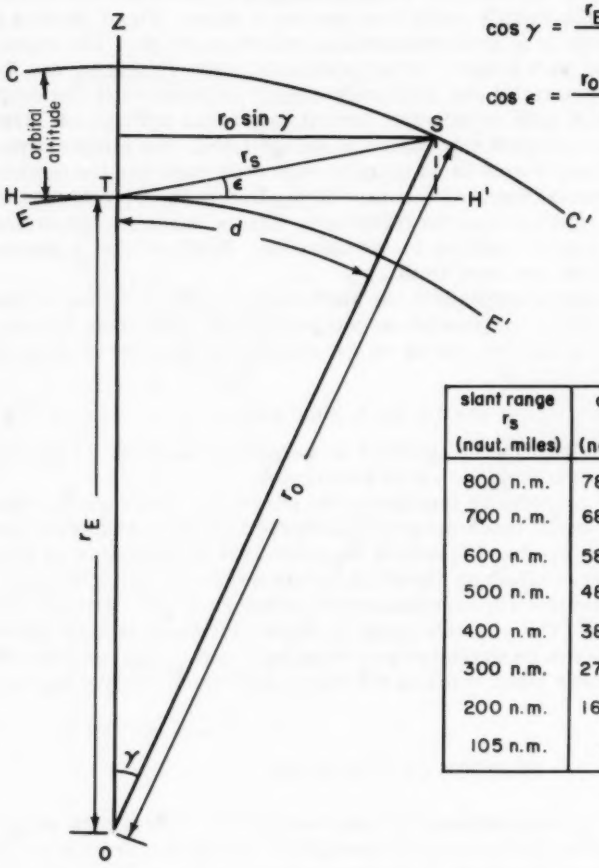


FIG. 2.—REFERENCE FRAME

at which the spacecraft will first appear as it comes over the horizon. This is termed the acquisition azimuth. Similarly, the direction to the point at which the "target" will disappear is called the departure azimuth.



$$\cos \gamma = \frac{r_E^2 + r_0^2 - r_s^2}{2\, r_E\, r_0}$$

$$\cos \epsilon = \frac{r_0 \sin \gamma}{r_s}$$

slant range r_s (naut. miles)	distance d (naut. miles)	elev. angle ϵ (degrees)
800 n.m.	782.5 n.m.	0° 47'
700 n.m.	682.4 n.m.	2° 53'
600 n.m.	582.3 n.m.	5° 9'
500 n.m.	481.7 n.m.	8° 2'
400 n.m.	380.1 n.m.	12° 3'
300 n.m.	276.8 n.m.	18° 28'
200 n.m.	167.5 n.m.	30° 19'
105 n.m.	0	90°

E-E' - EARTH'S SURFACE, C-C'-CIRCULAR ORBIT, T-TRACKING STATION
H-H'-HORIZONTAL THROUGH T, O - GEOCENTER, S - SPACECRAFT,
r_E - RADIUS OF EARTH, Z-ZENITH,
r_0 - RADIUS OF ORBIT = $r_E + a$, r_s - SLANT RANGE,
ϵ - ELEVATION ANGLE, d - DISTANCE ALONG EARTH'S SURFACE

FIG. 3.—DETERMINATION OF RANGE AND ELEVATION ANGLE

A spacecraft in circular orbit, traveling 18,000 mph, at 105 nautical miles altitude, will cross from horizon to horizon in approximately 5 min, if it passes directly overhead. In the more likely eventuality that it passes below the

zenith, the time will be even shorter. Refraction of light, infrared, and radar beams may permit acquisition at a point slightly below the geometrical horizon. Knowing in advance where the spacecraft will come into range near the horizon reduces the amount of scanning necessary and saves time in acquisition.

Plotting the azimuth graphically on a map is subject to the distortion inherent in projections of the earth's curved surface on a plane. Fig. 2 depicts a nominally circular orbit in a three-dimensional reference frame. The reference frame is inertial with respect to the prelaunch node. Orienting the X-axis toward this point permits the relatively simple expression of the orbit given in Eq. 1. If the X-axis is oriented toward the vernal equinox, or First Point of Aries, a more complex expression is necessitated. The simple equation based on the prelaunch node as an inertial reference point has the further advantage that differential corrections may be applied to it in evaluation of actual tracking data. Corrections for oblateness effects, such as regression of the orbit plane,[4] may be applied to the equation. However, for a short-duration trajectory these are very small.

Knowing the latitude and longitude of the station and of various points on the orbit from the preceding, it is possible to compute the azimuth from the station to the spacecraft at various points on the track using the law of cosines for sides:

$$\cos a = \cos b \cos c + \sin b \sin c \cos \alpha \quad \ldots\ldots\ldots\ldots \quad (2)$$

Fig. 3 shows the trigonometry involved in computing the slant range and elevation angle to the spacecraft at various distances.

There are several corrections familiar to the surveying civil engineer that are applicable in spacecraft tracking: geodetic corrections for curvature of the earth, oblateness of the spheroid (geodetic measurement of oblateness as distinct from the oblateness effect on the orbit, mentioned previously), and elevation differences; gravimetric corrections for deflection of the vertical, and refraction corrections. The probable range of values for these factors under the trajectory criteria can be evaluated mathematically and compared with the accuracy of the radars or other tracking devices to determine relative significance.

HORIZON CLEARANCES

It is generally desirable to situate tracking instruments where there are no obstructions to following the spacecraft throughout its full arc from horizon to horizon. Mountaintop locations, that offer a clear sweep of 360°, are not always feasible because of climatic conditions, difficulties in procuring property, poor accessibility, remoteness from developed areas, or the higher costs that are associated with these features.

One step in investigating a possible site is to determine the effect of any obstructions to tracking. This is done by reading the azimuth and vertical angle to prominent terrain features around the horizon with a transit. Magnetic azimuths are generally adequate for this purpose, provided declination corrections are made. Particular attention should be paid to nearby hills or buildings as well as to mountains, valleys, plains, and coasts. Where it is pos-

4 "Research Studies Related to Mapping Geodesy, and Position Determination," Midwest Research Inst., 4th Interim Tech. Report, June 19, September 18, 1959.

sible that elevating the tracking instrument on a tower may overcome a nearby interference, the distance to it should also be taken. Narrow obstructions such as flagpoles or lighthouses may be acceptable if they do not interfere with a length of arc greater than, for example, one or more widths of the radar beam measured between half power points.

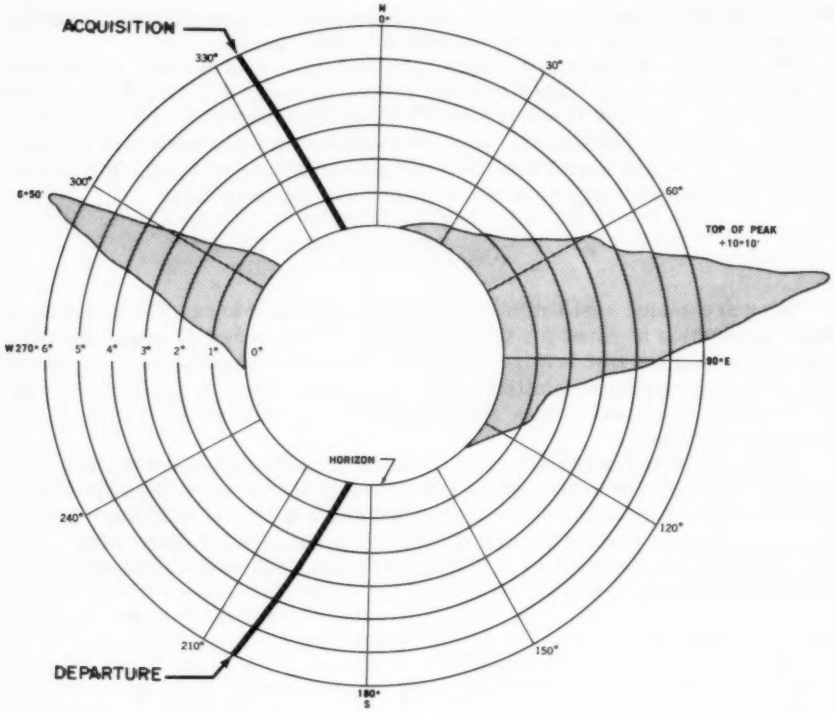


FIG. 4.—SHADOWGRAPH

The mask of the horizon is then plotted as a horizon profile or a shadowgraph and compared with the acquisition azimuth and departure azimuth. An example of a shadowgraph is shown in Fig. 4.

ROUGH LAYOUT AND PROPERTY SURVEYS

The layout of station facilities during site investigation is generally only approximate. Main purposes are to determine the extent and configuration of the area and its ability to meet criteria for orientation and separation of facilities. Orientation of tracking station facilities requires more attention than some other types of installation because of angular clearances, and adequate sepa-

ration of equipment is necessary to avoid electrical interference with radar and radio communications. A low level of background radio noise is particularly important when the spacecraft signal is to be followed to fade-out in deep-space tracking.

A property description sufficient to permit preliminary real-estate negotiation may also be required during the site selection phase of establishing tracking stations. In accordance with certain state laws and the recommendations of the ASCE task committee on surveying and mapping status, final property surveys should be made by a licensed land surveyor.

After a station site has been chosen, a topographical survey may be required for design engineering. Other surveys will be performed during construction and operation. Besides construction layout, as-built surveys, and the geodetic surveys mentioned previously, there are requirements for surveying peculiar to the dawning space age. These include bore-sighting surveys for directional radars, parallax computations, and evaluation of tracking data.

CONCLUSIONS

There are unique applications of surveying in space work. The procedures used in selection of sites for tracking space vehicles and in actual tracking require techniques that involve angle and distance measurements by optical and electronic methods familiar to the surveyor. This is particularly true where distance measurements are of earthly proportions rather than astronomical proportions.

Mapping may be facilitated by utilizing the mathematics of spherical geometry. In comparative evaluation of sites for tracking programmed spacecraft it is desirable to determine the acquisition and departure azimuths. Special requirements also pertain to geodetic and layout surveys for these sites. Further application of surveying techniques is required in the development of spacecraft tracking.

Journal of the
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Proceedings of the American Society of Civil Engineers

HIGHWAY AND BRIDGE SURVEYS: VERTICAL CONTROL AND SPECIAL CONTROL PROBLEMS IN BRIDGE CONSTRUCTION

Progress Report of the Committee on Engineering Surveying of the Surveying and Mapping Division

PART I—VERTICAL CONTROL

INTRODUCTION

The proper execution of a major water crossing is as dependent on precise vertical control as on precise horizontal control. The operations required for establishing and maintaining this control must be conducted with the care and precision prescribed for first-order work.

Because of the arbitrary manner in which a datum can be established, it is frequently found that many different datums were used to establish elevations of bench marks in a single locality.

In some extreme cases, as many as fifteen different datums have been used within a small area. For this reason, it is imperative to be certain of the datums to which existing bench marks are referred and to know the relative elevations of the various datums. Moreover, because accuracy in establishing bench marks and subsequent movement cannot always be determined, a situation rarely will exist in which it will not be necessary to establish a system of checking existing bench marks or setting new ones.

ESTABLISHING VERTICAL CONTROL

Establishing vertical control consists of determining precise elevations of existing or new bench marks at critical locations throughout the construction site. Leveling methods should incorporate first-order procedures, specifica-

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Surveying and Mapping Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SU 2, July, 1961.

tions, and instruments. Two methods of projecting levels over long distances, usually over water, require the use of the precise tilting level.

DIFFERENTIAL LEVELS

In most cases, establishing vertical control on land can be accomplished by the method of differential leveling, using first-order procedures. For best results, the length of sights should be limited to about 150 ft for ideal situations, and reduced as required under adverse atmospheric conditions. Differential levels should be run in closed circuits and adjusted between known bench marks. Specifications for observations, methods of adjustment, and sample notes for differential levels can be found in the ASCE Manual 10.[1]

In some instances where differential levels must cross ravines or streams requiring sights longer than normally permitted, the method of reciprocal levels should be used. Descriptions of this method of leveling can be found in most textbooks on surveying. It is designed primarily for use with one instrument; however, a more desirable procedure would be to make simultaneous observations with two instruments. After the first set of observations has been completed, the observers and instruments should be interchanged immediately and the observations repeated. This procedure minimizes any error introduced by a change in the atmospheric conditions. The mean value derived from the two sets of observations will be the true difference in elevation.

RIVER CROSSING METHOD

When it becomes necessary to extend vertical control over large bodies of water or onto distant piers a modification of the method of reciprocal levels devised by the United States Coast and Geodetic Survey (USCGS) as the river crossing method, can be used. This procedure is described in a USCGS Manual.[2]

TRANSBAY LEVELS

In order to establish vertical control for the San Francisco-Oakland Bay Bridge, sight distances of 10,300 ft were required. Establishing vertical control over this distance was made possible by the method of transbay levels, which was developed from the river crossing method of the USCGS to be used at night. It is essentially a modification of the vertical angle method of leveling, into which the absolute value of the angle does not enter.

The success of this method of leveling depends almost entirely upon using a target with exceptional definition at this distance. After extensive experiments, a target consisting of two flashlight bulbs placed one above the other was adopted. Two of these targets are mounted on the leveling rod so that the distance between them is equal to 1/2 ft for each 1,000 ft of sight distance. The bulbs on the target would be separated by about 1/12 of the target interval. When observing the target the cross hair should be centered between the two lights (Fig. 1).

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1 "Technical Procedure for City Surveys," ASCE, Manual 10.

2 "Manual of Geodetic Leveling," by Howard S. Rappleye, Special Publication No. 239, U.S.G.P.O., 1948, p. 51.

In making transbay level observations, the micrometer screw is read with the instrument successively sighted on the upper target, in the level position, and on the lower target. The intersection of the level line of sight on the transbay rod is computed by proportioning the rod interval between targets in the same ratio as that of the difference between the micrometer readings. The rod reading at the level line of sight on the transbay rod is combined with the rod reading at the level line of sight on the near rod to obtain the difference in elevation between the bench marks. In order to attain maximum precision, two instruments should be used and observations taken simultaneously in both directions. This procedure compensates for the curvature of the earth and minimizes any error due to a change in the atmospheric conditions. Instru-

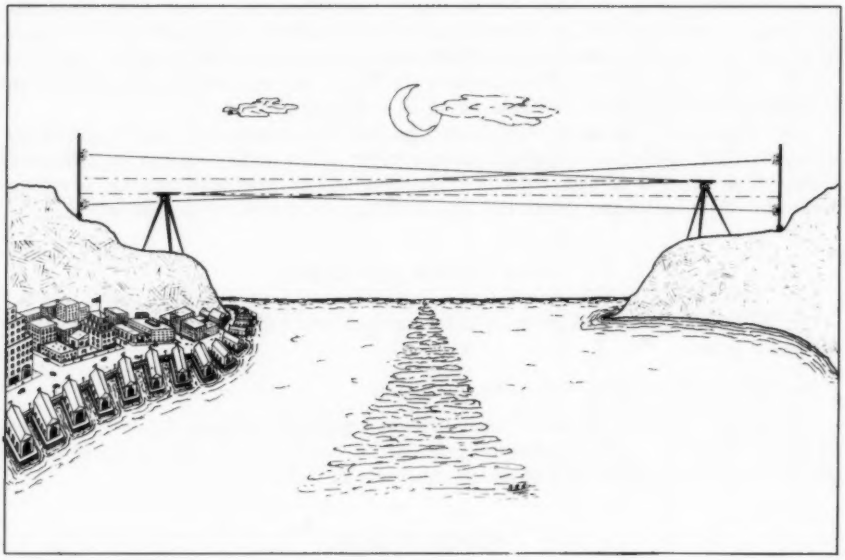


FIG. 1.—METHOD USED FOR PROJECTION OF TRANSBAY LEVELS

mental and personal errors are minimized by repeating the observations with the observers and instruments interchanged.

A set of simultaneous observations consists of reading the near rod at both setups and subsequently taking a predetermined number of readings with the instruments sighted in each of the three positions on the transbay rod. The observers with their instruments exchange locations and obtain a second set of readings, after which the instruments are moved slightly, then leveled, and a third set of readings obtained. The fourth set of readings is taken after the observers with the instruments have returned to the original location. These four sets of observations complete the series in the determination of a difference in elevation between the two bench marks. For first-order results, the

final difference in elevation should be taken as the mean of four sets of 25 readings. The number of readings at each setup may be reduced to ten when less accuracy is required.

In the early development of the method of transbay levels, numerous studies were made to determine the probable accuracy that might be attained under various conditions. Among those studies was one to determine the relationship between the number of observations and the average and maximum departures from the mean value for levels projected simultaneously in both directions. A similar study was also made for levels projected in one direction and corrected for curvature and refraction. The results of these studies are shown in Figs. 2 and 3. The data for plotting these graphs were obtained from the results of levels projected April 25, 1933, between Telegraph Hill Station in San Francisco and the Station, Goat, on Yerba Buena Island, a distance of 9,663 ft. The figures in the graphs indicate the number of groups used.

The observations in Fig. 2 were taken simultaneously from both stations, and are not complete, as the observers and instruments were not interchanged. However, the results shown are indicative of the accuracy that may be expected from levels projected in this manner. In Fig. 3 the curvature and refraction is taken as 4/5 of the theoretical value of curvature alone.

The results of transbay levels on the San Francisco-Oakland Bay Bridge indicated that a series of observations taken after darkness and on different nights gave results comparable to those obtained by standard precise leveling methods. The following gives the specifications for first-order leveling.

FIRST ORDER LEVELING

a. Average spacing of permanently marked bench marks along lines, not to exceed 1 mile.

b. Length of section, 1/2 mile to 1 mile.

c. Check between forward and backward running between fixed elevations or loop closures, not to exceed 4 mm $\sqrt{K}$ or 0.017 ft $\sqrt{M}$. The term K is the distance in kilometers, and M is the distance in miles.

BENCH MARKS

Widespread construction activities for most major water crossings require extensive nets of accurate bench marks from which elevations for construction purposes can be obtained. Once the basic datum has been established on each shore, precise-level circuits must be run throughout the areas of construction. The proper coordination among the various bridge units under construction at separate locations is dependent upon unified, reliable, and convenient vertical control at every site and during every phase of construction. The bench mark location, then, must be carefully chosen. Some of the factors to be considered are convenience to construction operations, safety from disturbance by these activities and freedom from subsidence and other natural defects. Since the possibility of subsidence is always present, frequent checks must be made on all bench marks in order to insure their accuracy. Both the initial establishment and the checking of bench marks require precise leveling methods and techniques. Special bench marks for the purpose of checking settlement are

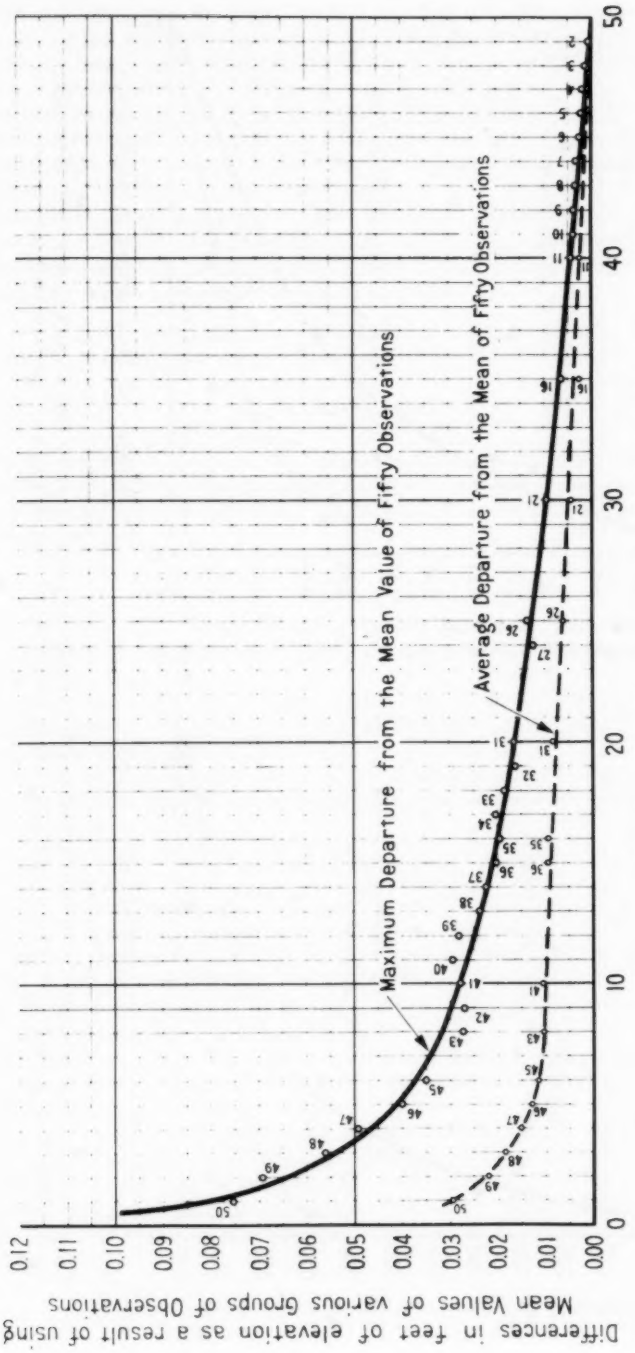


FIG. 2.—DIFFERENCES IN ELEVATION AS A RESULT OF USING MEAN VALUES OF VARIOUS GROUPS OF OBSERVATIONS

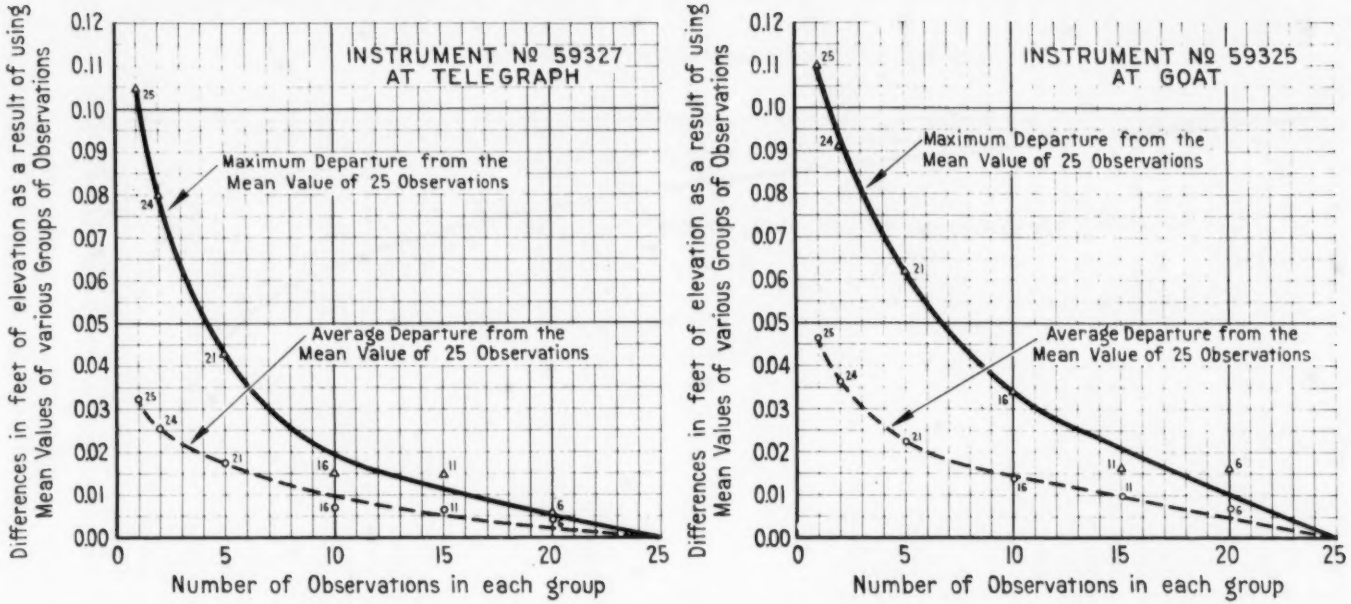

INSTRUMENT Nº 59327
AT TELEGRAPH
Maximum Departure from the
Mean Value of 25 Observations
Average Departure from the
Mean Value of 25 Observations
Differences in feet of elevation as a result of using
Mean Values of various Groups of Observations
Number of Observations in each group
INSTRUMENT Nº 59325
AT GOAT
Maximum Departure from the
Mean Value of 25 Observations
Average Departure from the
Mean Value of 25 Observations
Differences in feet of elevation as a result of using
Mean Values of various Groups of Observations
Number of Observations in each group

FIG. 3.—DEPARTURE IN ELEVATION RESULTING FROM USE OF VALUES OBTAINED FROM VARIOUS GROUPS OF SINGLE OBSERVATIONS PROJECTED IN ONE DIRECTION AND CORRECTED FOR CURVATURE AND REFRACTION

sometimes set in earth fills to check the progress of compaction over a period of time.

During the early construction stages of offshore piers, temporary bench marks can be set on nearby survey platforms, construction docks, or cofferdams. These bench marks should be checked frequently and should be sufficiently accurate for control of the under water construction. Temporary bench marks should be established on the piers as early as possible and their elevation accurately determined. When the pier has been completed, bench marks should be established at each corner for grinding the seats or setting base plates, and for determining settlement of the pier.

If possible, observations for pier settlement should continue over the years or until such time that it becomes apparent that the rate of settlement has decreased to amounts that can no longer be determined. These records may give valuable information for designing future projects.

PART II—SPECIAL CONTROL PROBLEMS IN CONSTRUCTION

Suspension Bridges

Suspension bridges account for many unusual control problems. Among them are the control required for erecting the main cable, grinding the inclined bearings for the cable bents, and boring the inclined cable anchorage tunnels where they are used.

CONTROL FOR SUSPENSION CABLES

The problem of correctly adjusting the main cables to design position is a challenging one. This problem involves some special computations and is complicated by the necessity of considering the frequent changes in the position of the various elements of the structure. Computations for this phase must begin well in advance of the start of construction.

The method as used during the cable spinning operations on the San Francisco-Oakland Bay Bridge will be explained in detail as one of the methods that can be used for guide-wire control.

The portion of the San Francisco-Oakland Bay Bridge spanning the 8,270-ft channel between San Francisco and Yerba Buena Island consists of two suspension bridges having a common center anchorage. These bridges have central spans of 2,310 ft and side spans of 1,160 ft. The vertical clearance, to provide for navigation, varies from 187 ft at the pierhead line to 227 ft at the center anchorage. Each of the cables for the suspension spans contain 17,464 parallel wires built up into 37 separate strands and compacted into a circular shape 28.75 in. in diameter.

The problem of correctly adjusting cables to their designed positions was unusual, due in part to some of the features that distinguished the paired spans from other suspension bridges.

The tower heights and cable sags varied to conform to the long vertical curve of the twin bridges. The mid-span elevations of the north and south free cables differed because of the unsymmetrical design required by the operation of interurban trains at one side of the lower roadway. These conditions necessitated fourteen separate control systems for the cable location. The location

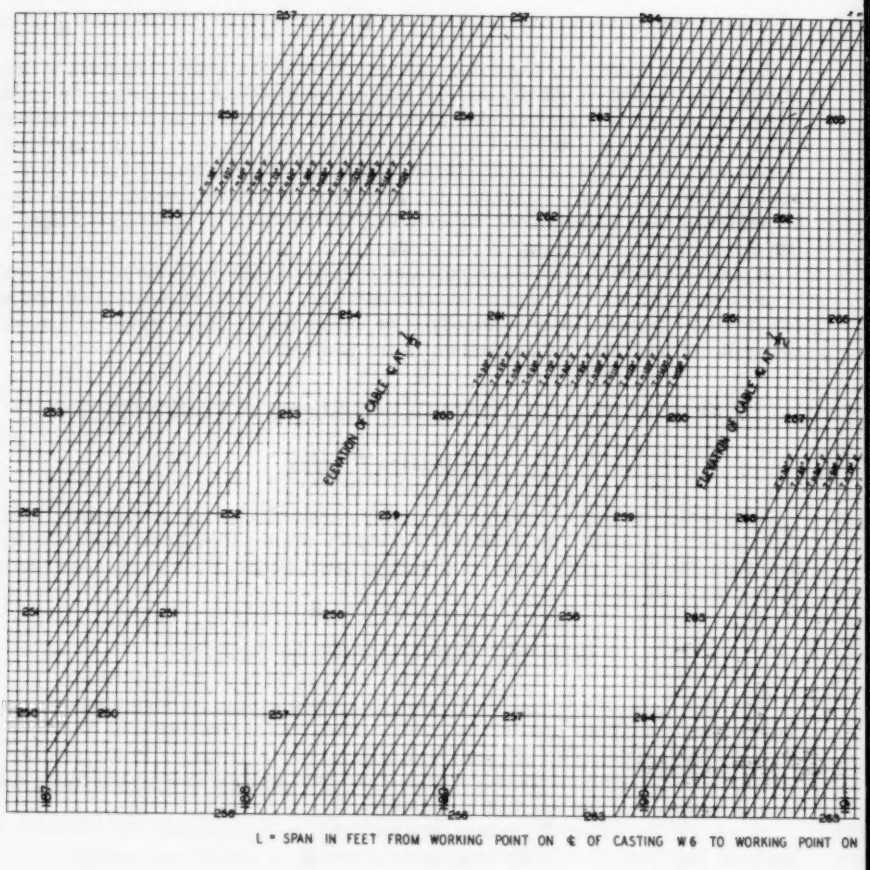
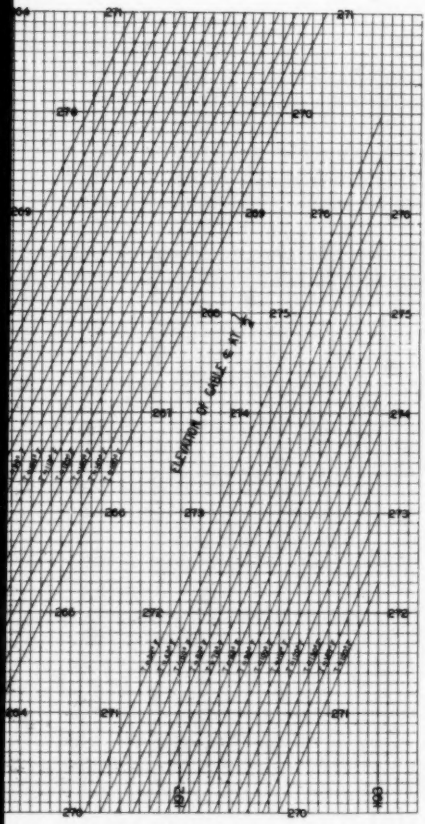
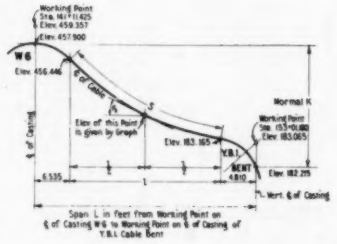


FIG. 4.—CABLE BENT ELEVATIONS AT MIDSPAN OF NORTH

of the San Francisco anchorage made necessary an 830-ft backstay span, which in turn required a rocker bent to support the cables at Pier W-1. The sags of the backstay cables decreased 19 ft during bridge construction. This movement, combined with sag changes and elastic elongation in the span between Pier W-1 and Pier W-2, made it necessary to move the cable saddles at Pier W-2 westward 4.53 ft during cable spinning to prevent excessive stress on Tower W-2. To further complicate this problem, the cables were spun in their final position at the tower saddles and in temporary strand forms at the anchorages.

In preparation for this work, the length of the cables in each span was computed from the funicular polygon of the loaded structure and reduced to a free cable length by removing the elastic elongation due to the suspended loads. The curve of the free cables was assumed to be a catenary and its equation was used to compute the mid-span sags, as the parabolic equations commonly used on shorter-span suspension bridges did not give results of sufficient accuracy.





NOTE: Given conditions are:
E = 28,000,000 lbs. per square inch
W = 490 lbs. per cubic foot
A = 521.56 square inches
L = 1178.410 ft. between points of tangency
S_1 = 1216.148 ft. " " " " "
S_m = 1216.280 ft. (Length at no stress and 62° F.)
h = 275.281 ft. between points of tangency.
θ_1 = 0° 58' 46"
Normal span between Working Points = 1189.755 ft.
Horizontal distances between points of tangency and Working Points considered constant.
Normal temperature = 62° F.

The values here shown are for the centerline of the north cable at midspan. To obtain the elevation at midspan of any other element in the cable cross-section, add algebraically the difference in elevation of that element and the theoretical center of the cable, (457.900), at the bottom support.

When the difference in elevation of supports, (K), is other than 275.685 ft. use additional increment changes in midspan elevation as shown on Dwg. № 66

To use these graphs for other than the above value of S_m (1216.280), an algebraic correction must be made to the temperature values here shown, equivalent to the following: $\frac{S_m - 1216.280}{0.00184500S}$ (where S_1 is the unstrained wire length, at 62° F., between Working Points, minus 11.648 ft.)

To adapt this graph to the determination of elements of the north cable other than the centerline, and likewise to utilize it for the south cable elements, the temperature range has been extended and differences in elevation have been correlated in degrees of temperature.

CABLE FOR VARIOUS SPANS AND TEMPERATURES

The office problem was to prepare graphs and tables so that the cables could be adjusted to correct position by setting the mid-span point at the proper elevation for any reasonable combination of temperature, span length, and change in elevation of supports.

When making computations for the graphs, consideration was given to the elastic deformation in the cable length Fig. 4 gives the elevation of the cable at the mid-point of the span resulting from different span lengths and temperatures. Several points were computed by the catenary formula and the curves interpolated between these points. The mid-span elevations were computed for the catenary between points of tangency at the tower saddles. To simplify the use of the graphs, the abscissa was made the equivalent span length as measured horizontally between vertical scribe lines on the center lines of the castings. Fig. 5 gives corrections to the mid-span elevation of the cables resulting from differences in elevations of the cable supports, for various span lengths. (Fig. 4(b) and the notes for the "given conditions" apply for Fig. 5.) By com-

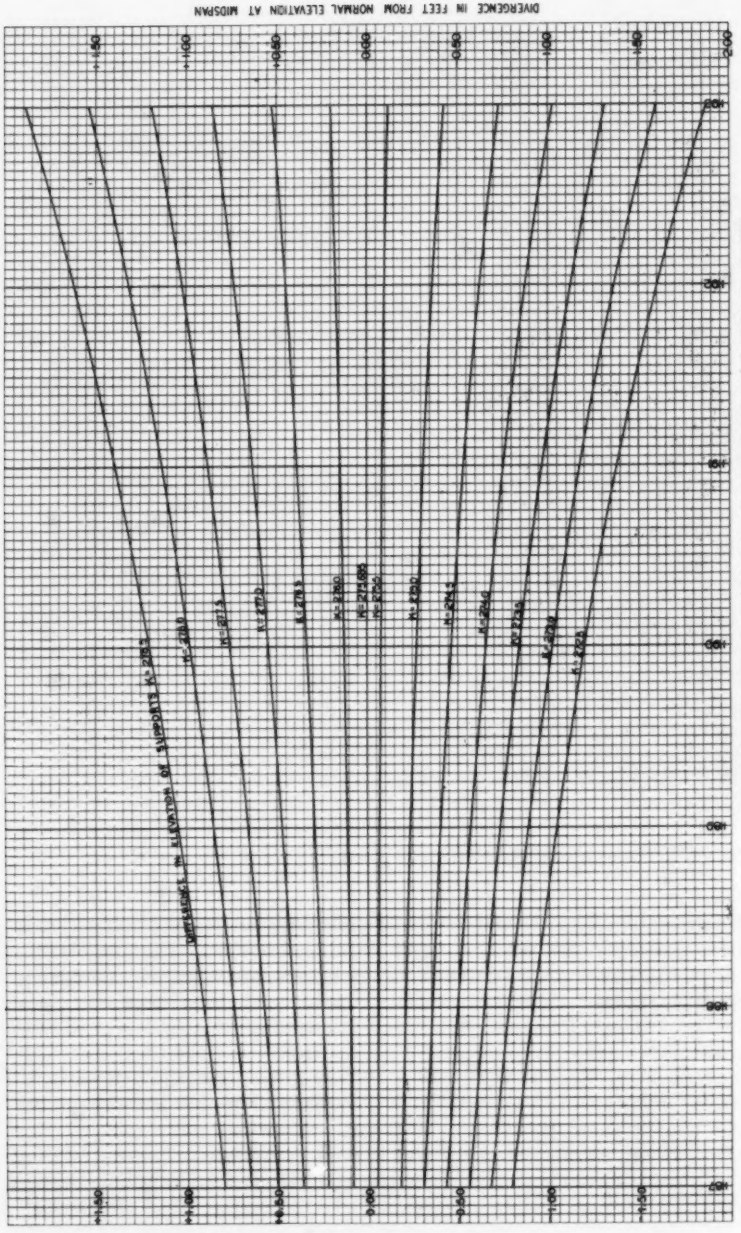


FIG. 5.—CABLE BENT DIVERGENCE FROM NORMAL ELEVATION AT MIDSPAN RESULTING FROM CHANGES IN RELATIVE ELEVATIONS OF SUPPORTS

bining the two graphs in Figs. 4 and 5 the correct mid-span elevation of the cables was obtained for any combination of span length, temperature, and difference in elevation of supports.

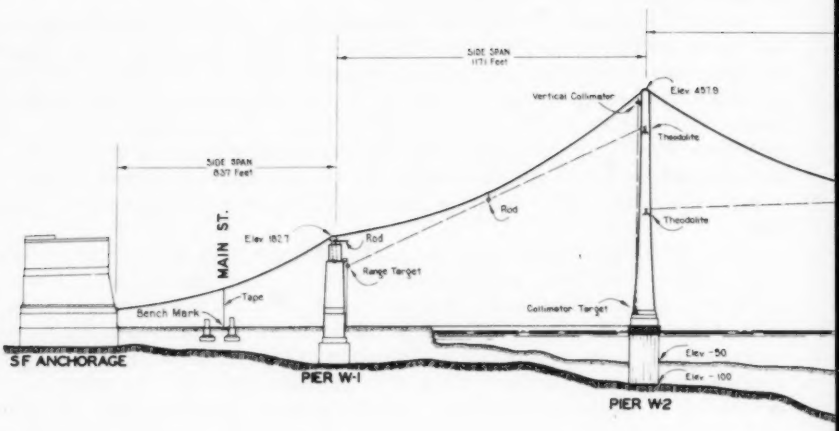
Figs. 4 and 5 were prepared for the centerline of the north cable and adapted for use in setting the strands of both cables by applying temperature constants. A temperature constant was the divergence from the observed temperatures at which the length of the centerline of the north cable became equal to the length of the strand. These temperature constants, ranging from plus 67.2°F to minus 30.0°F were added to the observed temperature to obtain an equivalent temperature with which to enter the tables. Data from these graphs were compiled in tables and bound in pocket-sized volumes for convenient use in the field.

Two additional tables were required at Pier W-1 due to the slope of the rocker bent, which was rotated as much as 11° from the vertical during cable spinning. Any rotation of the bent produced a change in the station and elevation of the working point. The first table listed the elevations corresponding to various stations of this point. Due to the slope of the rocker bent, the span lengths of the strands vary according to their position with reference to the centerline of the cable. The span length was first computed from the observed station of the working point and corrections, listed in a second table, were applied to this length to obtain the span lengths for the various strands.

In preparation for field work, computations were necessary to determine the positions of the control points for locating the cable. The normal positions of the range targets were computed for a line of sight paralleling the tangent to the cable at the midpoint. The line of sight parallel to the tangent minimized the errors resulting from inaccuracies in the location of the midspan point (Fig. 6.)

The elevation of the cables for any condition could be theroretically computed to any degree of accuracy, but the limiting factors in the field were the precision with which the functions, such as span length and temperature, could be measured. To evaluate these factors, a study was made on a 500-ft model of cable wire. The results indicated that thermometers shrouded in short pieces of cable wire gave the most accurate measurements of the temperature of a guide wire and that wind and condensation of moisture had a negligible effect on the sag. To determine the rate and extent of tower deflections resulting from differential temperatures in the tower legs, the locations of the tower tops were observed with a vertical collimator throughout a 24 hr period and, at the same time, temperatures were taken of the tower steel. Differential temperatures as large as 40°F, and deflections in the towers of nearly 1 ft parallel to the bridge centerline and less than 1 in. normal to it were observed (Figs. 7 and 8). The results of the observations indicated that all towers lean from 1 in. to 2 in. east of their design position. It was also found that wind produced vibrations but no measurable deflections.

To made instrument observations and adjust the range targets, platforms were built at desired elevations on the towers. To protect the instruments and observers these platforms were roofed and enclosed in canvas. The mid-span rods and range targets were illuminated for night work and the theodolites for these surveys were equipped for electrical illumination of the cross hairs. At Pier W-1 a rod graduated to give directly the station of the saddle casting was attached to the rocker bent. Before cable surveys began the stations and elevations of cable saddles, temporary splay castings, instrument and target mount-



NOTES

At each tower and at midspan Fahrenheit thermometers, encased in cable wire, are to be placed parallel to the cable.
Instruments, cross hairs and verniers shall be electrically illuminated.
Instruments shall be set at a predetermined elevation.
Rods and targets shall be defined by red and white electric bulbs.
Rods at midspan are to be of constant length.
The range targets shall be adjusted to line-on the rod at midspan to the desired elevation.
Communication between control points of each span, will be by telephone.

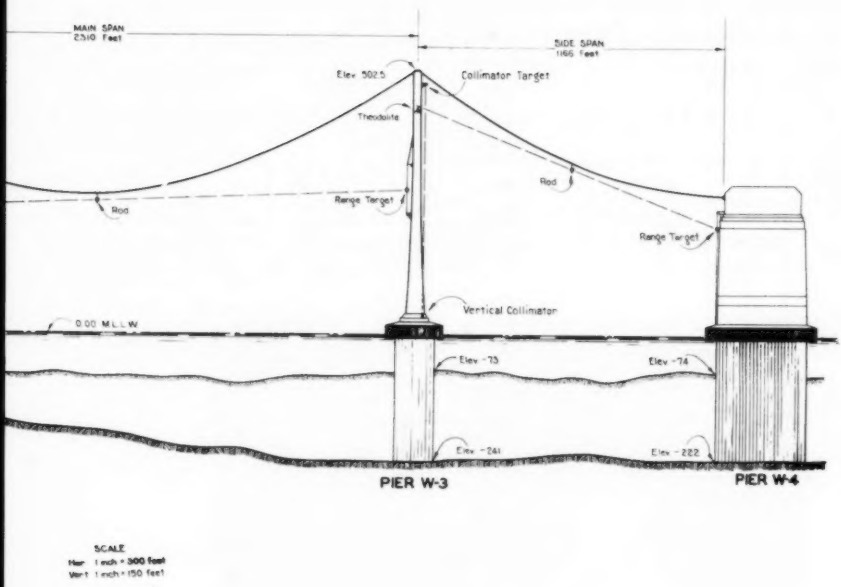
FIG. 6.—INSTRUMENT ARRANGEMENT

ings were determined, checked and referenced. Mid-span points were established by triangulation.

Communications for survey control on each of the two bridges were maintained by magneto telephone circuits. Permanent telephones were placed at the instrument and range target stations, and plugs for portable telephones were placed at other important points. These telephone circuits, complete for each span, could be connected in series and used as a single unit. Each circuit could also be connected to the trunk line, which extended to the main bridge office.

The field personnel for guide-wire and strand adjustment consisted of 12 men divided into two parties. By using two parties, the survey operations were always abreast of the adjustments, as one party lined in the cables in one span while the other party moved into position for adjustment of the adjacent span.

All observations on guide wires or strands were taken at night, for at that time the temperatures of the wires could be more accurately measured and the rate of movement of towers and rockers was at a minimum. Temperatures



SCALE
Hor 1 inch = 300 feet
Vert 1 inch = 150 feet

FOR ALINEMENT OF CABLES

were observed at the mid-span and near each of the towers. Guide-wire temperatures were obtained by inserting in the strand thermometers in metal cases.

To determine variations in span length, it was necessary to know the station of the cable saddles at all times during adjustment. This information was provided by continuous observation taken with vertical collimators. The elevations of the cable supports were obtained from bench marks adjacent to the castings. The station and elevation at Pier W-1 were determined from the reading of the special rod previously described. The mid-span rod was hung vertically downward through the catwalk with the zero of the rod on the wire to be adjusted. The theodolite was sighted at this rod and the range target was lined in.

With the collected field data and the tables, the correct elevation of the mid-span point was obtained and the corresponding position of the range target computed. The divergence of the wire from the theoretical position was obtained from the difference between the actual and computed range target settings. A graph was provided giving the change in length of the wire necessary to obtain the required sag correction. After the adjustment at the supports to correct

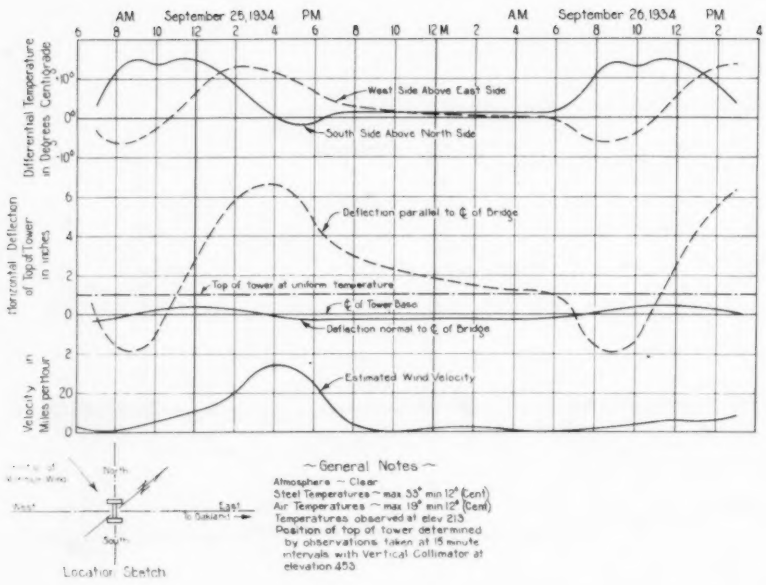

AM September 25, 1934 PM
AM September 26, 1934 PM
Differential Temperature in Degrees Centigrade
West Side Above East Side
South Side Above North Side
Horizontal Deflection of Top of Tower in Inches
Deflection parallel to ℄ of Bridge
Top of tower at uniform temperature
℄ of Tower Base
Deflection normal to ℄ of Bridge
Velocity in Miles per Hour
Estimated Wind Velocity
Location Sketch
North
East
To Oakland
South
~ General Notes ~
Atmosphere ~ Clear
Steel Temperatures ~ max 33° min 12° (Cent)
Air Temperatures ~ max 19° min 12° (Cent)
Temperatures observed at elev 213
Position of top of tower determined by observations taken at 15 minute intervals with Vertical Collimator at elevation 453

FIG. 7.—DIFFERENTIAL TEMPERATURES AND TOWER DEFLECTIONS, TOWER W-2

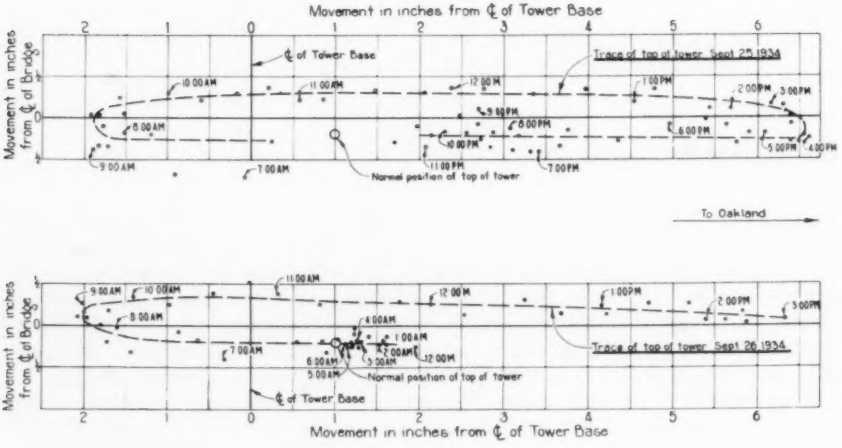

Movement in inches from ℄ of Tower Base
Movement in inches from ℄ of Bridge
℄ of Tower Base
Trace of top of tower Sept 25 1934
Normal position of top of tower
To Oakland
Trace of top of tower Sept 26 1934
Normal position of top of tower
General Note
Position of top of tower determined by observations taken at 15 minute intervals, with Vertical Collimator, at elevation 453.

FIG. 8.—MOVEMENT AT TOP OF TOWER W-2 RESULTING FROM DIFFERENTIAL TEMPERATURE IN TOWER LEGS

TABLE 1.—WEST SUSPENSION BRIDGE CABLES DIVERGENCE BETWEEN MEASURED LENGTH AND DESIGN LENGTH S.F. ANCHORAGE

Cable Condition (1)	to W-1 (2)	W-1 to W-2 (3)	W-2 to W-3 (4)	W-3 to W-4 (5)
North Free	+0.026 ft	+0.006 ft	+0.002 ft	-0.002 ft
Free		+0.025	-0.007	-0.002
Compacted		+0.019		+0.008
South Free	-0.005	+0.122	-0.209	0.000
Compacted		+0.104	-0.144	+0.006

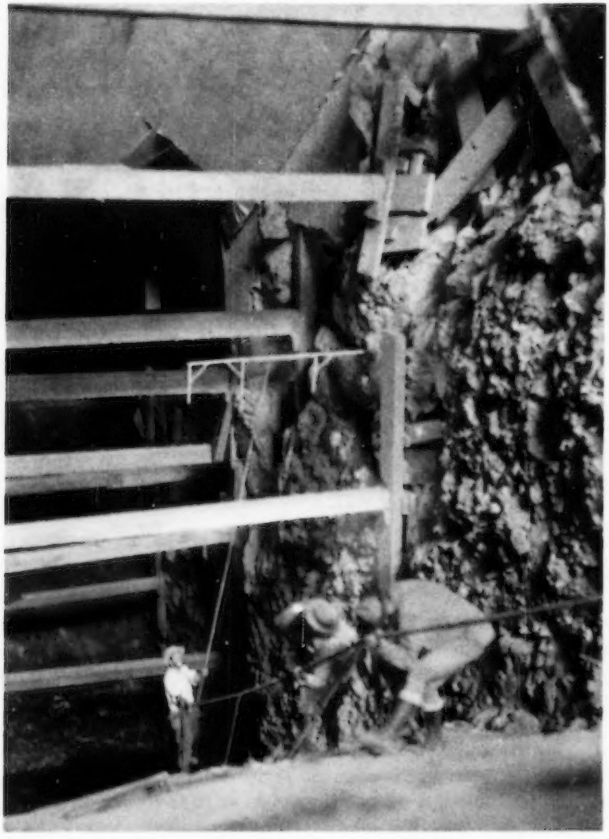


FIG. 9.—NORTH ANCHORAGE TUNNEL

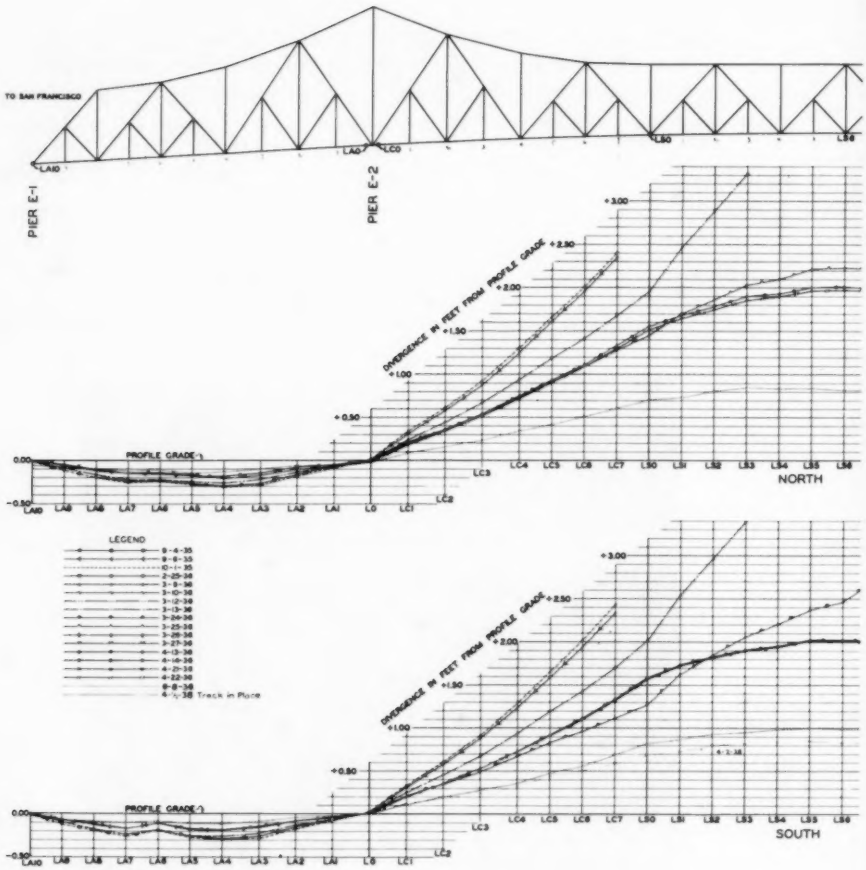
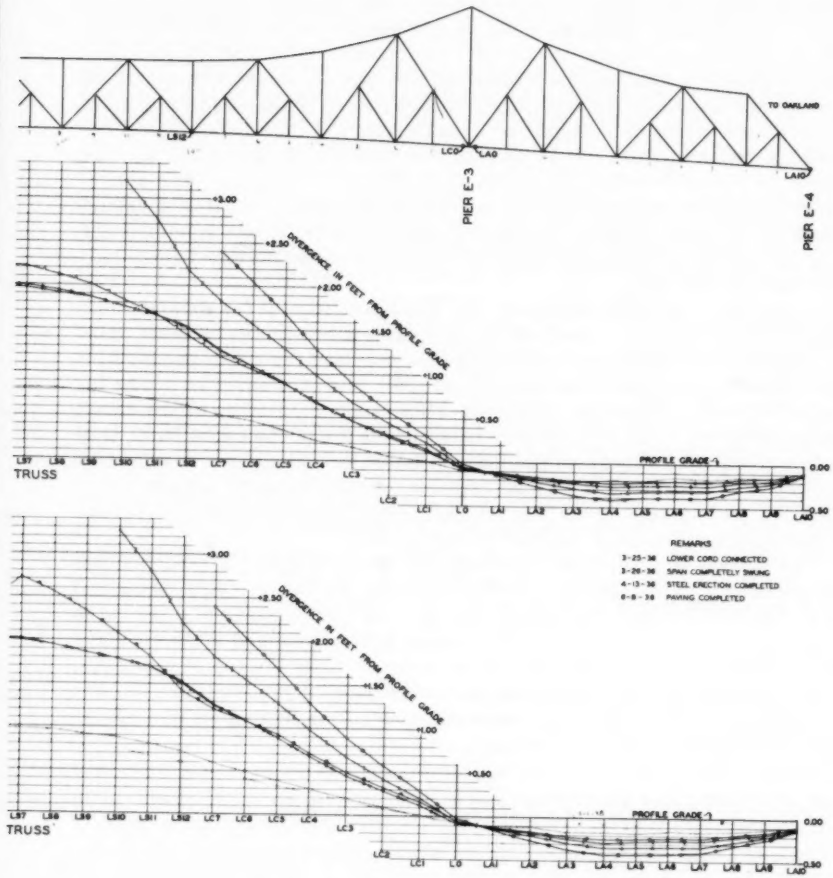


FIG. 10.—RECORD OF CAMBER

the error in sag was made, the sag was checked and, if necessary, additional adjustments were made to bring the wire to the correct elevation. Although the guide wires for a tier of strands were carefully set to the same elevation at the mid-span points, the wires sometimes diverged as much as 1 ft at the quarter-span points. The guide wires for the first strands were checked several times to insure their accuracy. The length of each strand in the cable was computed and tabulated and guide wires for subsequent strands were set by applying increment corrections to the length of the original guide wires. To verify these corrections, the sag of the guide wire for each strand was checked by field observations.

The first-tier strands were adjusted in the same manner as the guide wires. Only one strand in each succeeding tier was set by instrumental control and the other strands adjusted to it, using a spirit level and allowing for temperature variations. The adjustment of the controlling strand in each tier was



MEASUREMENTS, CANTILEVER BRIDGE

checked by measurement to the inside bottom strand of the cable, noting the differences in temperature.

At the completion of a strand adjustment, the temperature of each accessible strand was taken and its elevation determined by measuring from the bottom strand. The elevations of the strands were then reduced to normal conditions and recorded for reference.

Shortly after construction of the cables had begun, it was noted from this record that the mid-span elevations of the completed strands in the side spans were decreasing, and those in the backstay span were increasing. Investigatory surveys disclosed a deflection in the top of Pier W-1 of 0.05 ft to the east, due to the horizontal thrust of the inclined rocker arm, and similar surveys at top of Pier W-4 disclosed movement to the west. The movement of Pier W-4 was determined from the results of precise levels over the bench marks on the corners at the fender and at the roadway level, and was substantiated

by collimator observations taken on the center wall of the pier. The deflection at Pier W-4 increased as cable spinning progressed and amounted to 0.15 ft to the west at the time of completion of the west cables. However, at Pier W-1 no further movement was noted. These deflections accounted for nearly all divergence in the sags and the remaining changes were explained by temperature observations taken throughout the cross section of the cable. These observations indicated that the inside strands did not cool off as rapidly at night as the outer strands (sometimes being as much as 10°F higher) and consequently were partially resting on those beneath them with a resulting decrease in elevation of the lower strands. In setting later strands, it was assumed that 2/3 of the movement at Pier W-4 would be overcome when the east cables balanced the pull on Pier W-4.

Upon completion of cable spinning, the final positions of the strand shoes were checked and reference points for placing cable bands and permanent splay castings were set. To define the location for seizings and for cable bands, surveys were made along each cable before and after compaction. The points for bands were located by measuring with a steel tape supported on top of the cable. The centerline of the cable was marked to define the top of the cable band, thus preventing errors resulting from the daily twist of the cables due to differential temperatures. Table 1 gives the results of measurements made with a steel tape, on the cables of the west bridge. The table gives the divergence between the measured lengths and the design lengths.

Upon completion of the cable spinning, the final positions of strand shoes must be checked and reference points set for the placing of cable bands and permanent splay castings. To define the location for seizings and for cable bands, surveys must be made along each cable both before and after compaction. The points for bands are located by measuring along the top of the cable. During the process of cable spinning, and thereafter, it is necessary to constantly check the deflection of the towers in order to detect excessive movement, should it occur.

CONTROL FOR INCLINED BEARINGS

The cable bents or saddles located at the end of the side spans are supported on inclined bearings. These bearings must be ground within specified limits, and establishing control for this operation presents an unusual problem.

One method for controlling the grinding employs transit points and chisel-pointed bench marks set at regular intervals along the upper and lower edges of the area to be ground. A transit is set over the upper point and the telescope brought into a plane parallel to the theoretical ground surface by sighting on a computed rod which is normal to the surface on the lower bench mark. Intermediate points on the concrete surface are brought to grade by intersecting the normal grade rod with the transit cross hair. The rod may be made normal to the line of sight by mounting a rod level at the required angle with a small metal or wood mounting bracket.

CABLE ANCHORAGE TUNNELS

In some instances suspension bridge cables are anchored in tunnels driven into rock formations rather than in gravity anchorages. These tunnels must

follow the alinement of the cable, which is generally inclined at a steep angle. The construction of these tunnels may be controlled by the usual surveying methods of establishing line and grade.

A modification of this method, which takes advantage of the relatively small size of the tunnels, utilizes a large-scale plan and elevation of the tunnel. This sheet should be completed in advance of construction Lines representing elevations and stationing along the bridge center line as well as coordinates referred to the center line of the tunnel should be superimposed on the drawings. To use this device the positions of the instruments are determined from known reference points and plotted on the drawings. The angles and distances required for establishing the control and locating the roof beams, spring line, and other features of the tunnel can be measured and scaled directly from these drawings, thereby saving the time required for computations.

In the place of cross sectioning the side walls after the excavation has been completed, it may be advantageous to make a vertical-plane "contour" map of each wall, using contour lines of equal distance from the centerline vertical plane of the tunnel instead of lines of equal elevation. This can be accomplished by a combination of stadia and offset methods. The transit is set up at a given distance from and parallel to the centerline. By sighting along this parallel line at a graduated horizontal rod fastened to the upper end of a pole (length of pole is determined by the cross-sectional size of the tunnel), the offset distance to the line of excavation can be determined (Fig. 9). The distance and elevation to the horizontal rod may be determined by stadia readings on a vertical rod, which is also fastened to the upper end of the pole. When the tunnel is completed, the steel grillage for the anchorage is carefully defined for both station and elevation.

CANTILEVER, ARCH, AND TRUSS SPANS

Cantilever, arch, and simple truss spans require very little control for erection after the shoes or bearings have been properly set. Their geometric shape and position is primarily determined by the fabricated lengths of the members and the elastic deformation. During the erection of these types of spans the elevation of the panel points should be checked at various stages in order that the theoretical and the actual deflected shapes of the structure can be compared and the jacking devices adjusted to bring the structure into the proper alinement for closure (Fig. 10).

Respectfully submitted,

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Committee on Engineering Surveying of the
Surveying and Mapping Division

Journal of the
SURVEYING AND MAPPING DIVISION
Proceedings of the American Society of Civil Engineers

SURVEYING AND MAPPING DIVISION

1926 - 1960

Report of the Chairman
Executive Committee

SYNOPSIS

This report outlines the history of the Surveying and Mapping Division of the American Society of Civil Engineers. It covers the events that gave rise to the formation of the Division, its early organization, and its principal activities up to the present time (1961).

From time to time, surveying and mapping has been referred to as the civil engineer's "bread and butter." There are times when the bill of fare is so varied and rich that little attention is given to such prosaic substances as bread and butter. And with the vast and varied expansion presently taking place in the field of engineering, there may be a tendency to overlook the contribution that such sciences as surveying and mapping are making in the space age. Nevertheless, fundamentally and traditionally, surveying and mapping remains a significant part of civil engineering. This traditional relationship becomes evident from time to time in rather unobtrusive ways. For instance, only recently an announcement of a tour of historic points in the vicinity of Washington, D. C., said in effect, "Be sure to visit the surveyor's office used by George Washington, America's first civil engineer." Also, illustrations of construction projects frequently have surveying instruments in the foreground, using them to identify the project as an engineering development.

Note.—Discussion open until December 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Surveying and Mapping Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SU 2, July, 1961.

Many changes, some of them quite revolutionary, are taking place in the engineering field. These changes have an impact on techniques, on education, on the ethics of private and public practice, and on the very nature of the various elements that comprise the field of Civil Engineering. In regard to the latter point, this Division, at the Society's request, has carried out in the last three or four years, an extensive reappraisal of the place of surveying and mapping in the civil engineering profession. A Task Committee of this Division made this appraisal, and its report and recommendations have been quite widely published. The findings of this Task Committee were accepted by the Society's Board of Direction at its Los Angeles meeting in February, 1959. The big job now is to implement these far-reaching conclusions.

In contemplating an appropriate course of action in regard to these matters, it is well to take a look back in order to plot the future course with greater certainty. This backward look might be likened to a surveyor taking a backsight in order to accurately prolong his course, or to depart from it according to prior conclusions or computations.

In 1922 the American Society of Civil Engineers authorized the formation of technical Divisions within the Society. By 1926, there were eight such Divisions. In that year, the Board of Direction approved a petition signed by over 90 members of the Society requesting the formation of a Surveying and Mapping Division. The petition was circulated largely through the efforts of the then Chairman of the Federal Board of Surveys and Maps, William Bowie of the U. S. Coast and Geodetic Survey. He was aided in the work of development by many people representing a fair cross section of public service, private practice, and education. Among them were C. H. Birdseye, Chief Topographic Engineer of the U. S. Geological Survey; W. N. Brown, a well-known private practitioner of Washington, D. C.; and Clarence T. Johnston, F. ASCE, of the Department of Civil Engineering, University of Michigan, Ann Arbor, Mich. These men all served as early officers of the Division. Bowie served continuously as Chairman of the Executive Committee until his death in 1940, a period of about 14 yr.

In this report the activities of the Division have been divided into 4 parts. The first part, 1926 through 1930, is called "the formative years"; and the second, third, and fourth parts simply the 1930's, 40's, and 50's.

THE FORMATIVE YEARS

During these years, the officers of the Division were concerned with defining the scope of the Division's field of activity. The Executive Committee sought the views of members of the Division as to the categories of work involved in the field of surveying and mapping in order to establish a basis for organizing technical committees. This exchange is summarized in the following quotations from circular letters sent to members of the Surveying and Mapping Division on September 25 and December 30, 1926:

> "The field of action of this new Division is very broad and it is desirable to decide at an early date what activities should be taken up at once and what should be left for the gradual growth and development of the Division. You have an intimate knowledge of the general needs of the engineering profession for certain investigational and standardization work along surveying lines. . .would you send in. . .a digest of what in

your opinion are the activities which should be immediately taken up by the Division?

"Many of the letters received in response to the first circular letter contained valuable recommendations which will be turned over to the proper committees as soon as they are organized. It is interesting to note the different things stressed in the various letters. A large number emphasized the importance of having better specifications for property surveys. . .

"Other letters pointed out the importance of better specifications for metropolitan surveys and for construction surveys. The need for a good topographic map over the entire country was mentioned by several. At least a half dozen branches or details of surveying were classed as being in need of investigation or standardization. . .

"The Executive Committee has made a preliminary outline of the work of the Division and has provided for the following committees:

1. Location and Construction Surveys. To include highway and railroad surveys and surveys for construction work, reservoirs, and also mining surveys.
2. Boundary Surveys. To include cadastral surveys, surveys of political boundaries and of riparian boundaries.
3. Topographic and Hydrographic Surveys.
4. Geodetic Surveys.

"It is evident at once that there is a chance for serious over-lapping between the various groups and also that some very important branches of surveying are not dignified by having a separate committee assigned to them. For instance, the subject of retracement surveys will require a large amount of work to develop it properly. Metropolitan surveys are also engaging the attention of a large number of engineers and their importance well warrants a separate committee. At the same time, if proper coordination is to be obtained in reports and specifications, the committees must follow a logical subdivision of the science of surveying. Subcommittees can be organized within the major committees to cover any particular part of the work that of committee."

This organization of the Division was augmented by other committees in the following years. In 1930, in addition to the four committees previously mentioned, there were:

1. A Special Committee on City Surveys.
2. A Special Committee on Third and Fourth-Order Triangulation and Traverse.
3. A Special Committee on Levels.
4. A Special Committee on Definitions of Surveying Terms.

All these committees were interested in improving techniques and standardizing practices within their field of interest. Some of the committees were preparing Manuals of Practice, and the Special Committee on City Surveys reported that the final draft of a Manual of City Surveys would be ready by the Spring of 1930.

By the beginning of the 1930's, committee structure had been established, and work on technical manuals was in progress. Reports indicate that membership in the Division exceeded 600.

THE 1930'S

During the early 1930's the organization was modified by the creation of a Committee on Aerial Surveying and Mapping. This apparently came about from a suggestion that a committee of the International Society of Photogrammetry be formed in the United States. The "Special Committee on City Surveys" was made permanent; the "Committee on Location and Construction Surveys" was discontinued; and a "Committee on Highway and Bridge Surveys" was formed. In the mid-1930's the following committees were formed: "Committee on Map Information Offices," "Committee on Surveying and Mapping Courses in Educational Institutions," and a "Special Committee on Land Surveys and Titles." The latter was formed for the purpose of studying and recommending proper legislation for the perfection of land titles. Also, the "Committee on Topographic and Hydrographic Surveys" was changed to "Committee on Topographic Surveys." After a short and uneventful existence, the "Committee on Surveying and Mapping Courses in Educational Institutions" was terminated because of the existence of similar committees of the Society.

Threats of over-lapping of activities arose with the growing complex of committees, which led the Executive Committee to advise the Committee on Control Surveys, and the Committees on City Surveys, and Third and Fourth-Order Triangulation and Traverse to fully coordinate their work on manuals. In regard to manual preparation, the Committee on City Surveys published Manual No. 10 in 1934. The Committee on Definitions of Surveying Terms was hard at work on a manual, which was published as ASCE Manual No. 15 in 1938. Varied reports came from other committees. The Committee on Highway and Bridge Surveys concluded that the existence of several State Highway Department manuals on the subject obviated the necessity of preparing an ASCE Manual. The Committee on Topographic Surveys reported an outline for a manual stressing specifications for maps and methods of testing. In connection with that report, it was suggested that the American Society of Photogrammetry, which had come into being in 1934, be requested to appoint a committee to cooperate with the Committee on Topographic Surveys.

Later the Committee on Topographic Surveys reported that the Manual in preparation by the American Society of Photogrammetry, and the Topographic Instructions of the U. S. Geological Survey would probably fill most of the present needs of the topographer; hence, the Committee's plans for a manual were deferred. It is interesting to note that these seem to be the sole references in the files to the creation and existence of the American Society of Photogrammetry, which has become, as is well known, a large and highly respected technical organization.

So much for organizational matters in the 1930's. It appears that the nature of the Division's activities during those years was quite definitely influenced by the economic conditions of the times. Much of the Division's activity during that period was directed toward increasing the amount of work available to those engaged in surveying and mapping. During this period these resolutions, among others, were passed and sent to Washington:

1. January, 1932: A resolution urging the completion of the levelling and triangulation nets of the United States.
2. January, 1933: A resolution urging the computation, adjustment and publication of control data of Federal agencies, that was then being held up for lack of funds.

3. January, 1933: A resolution urging appropriation of funds at State and Federal level to carry out a cooperative program of topographic mapping fully commensurate with the increasing need for maps throughout the country.

Further indications of the stress of the times are found in statements urging members of the Society to support the foregoing proposals by contacting legislative officials at all State and Federal levels. The Chairman of the Executive Committee of the Surveying and Mapping Division wrote several letters to the President of the United States, directing his attention to the need for maps and the opportunities for their economical production during the period of the depression. In addition to the actions of the Surveying and Mapping Division, the Society, in 1931, authorized a committee under Birdseye to cooperate with the American Engineering Council in a program to develop interest in extending the topographic mapping of the country in general accord with the provisions of the Temple Act of 1925 of the United States Congress.

At the close of the 1930's the following technical committees of the Division appeared to have been active: Aerial Surveying and Mapping, Control Surveys, Map Information Offices, Topographic Surveys, Land Surveys and Titles, City Surveys, Definitions of Surveying Terms, Highway and Bridge Surveys, Special Committee on Third and Fourth-Order Triangulation and Traverse, and Boundary Surveys.

Manuals on City Surveys and Definitions of Surveying Terms had been published and the Chairman of the Committee on Control Surveys reported that the manuscript of a Manual on Control Surveys was at Society headquarters awaiting publication. In addition to the important work of outlining standards and procedures in manual form, considerable work had been done to stimulate interest in the development of the national scheme of control surveys and in expediting the national program of topographic mapping. The Division had over 1000 members by 1940.

THE 1940'S

In the early 1940's there was much interest in the development of the organization that is now well-known as the American Congress on Surveying and Mapping. At a conference on surveying education, Committee VIII of the Civil Engineering Division of the Society for the Promotion of Engineering Education proposed that a forum of a National Congress on Surveying and Mapping be held and that it be sponsored by various existing organizations pertaining to surveying, including the Surveying and Mapping Division of ASCE. The Division heartily assented and the forum rapidly developed into a permanent organization. The Society became concerned as to whether the development of the American Congress on Surveying and Mapping indicated failure on the part of the Surveying and Mapping Division to properly represent the interests of the surveying community. The Division, after considerable study, pointed out that membership in the Congress was not limited to professional Civil Engineers and consequently, furnished an excellent medium for all who might be involved or interested in the science. It was further concluded that there was an ample field of activity for both organizations, and that close coordination would be mutually beneficial.

The Surveying and Mapping Division continued work on technical manuals, and the Committee on Control Surveys reported in 1940 the publication of ASCE

Manual No. 20, "Horizontal Control Surveys to Supplement the Fundamental Net."

The impact of World War II is evident in the accounts of the 1940's. The press of emergency war-time activity introduced diversities into what had developed as a fairly uniform pattern of activities. The Division showed interest in the national plan for control surveys and topographic mapping to strengthen our defenses. It received first-hand reports on the build-up and strengthening of such military mapping units as the 29th and 30th Engineers. A report was prepared by a committee of the Division on "Surveying and Mapping Courses in Engineering Defense Training." This was sent by the Society to the Commissioner on Education in Washington, D. C., no doubt to become part of a larger field of literature dealing with defense training. The report offered 4 courses to be completed in a 16-weeks period: topographic draftsman; stereoscopic operator; geodetic computer; geodetic observer. The stress of war was strongly evident in this recommended "cram session."

Although the 1940's indicate less progress in the exploration of techniques and procedures, and so forth, there was much activity on emergency measures prompted by the war and post-way problems. At the close of the 40's there were probably about 1400 members to 1500 members of the Surveying and Mapping Division. Active technical committees were: City Surveys, Control Surveys, Definitions of Surveying Terms, Highway and Bridge Surveys, Land Surveys and Titles, and Topographic Mapping and Photogrammetry.

Some of these committees had been reorganized in the late 1940's or were in the process of reorganization following the war. It had been decided that the Committee on City Surveys would take on the job of revising ASCE Manual No. 10, and a major revision was under way of the material for Definitions of Surveying Terms. The Committee on Control Surveys recommended in 1949 that the 1936 material on Third- and Fourth-Order Triangulation and Traverse not be published by the Society. This recommendation arose entirely from the fact that 13 yr had been allowed to elapse since the material had been prepared and, although the work had been competently done by able engineers, progress in survey methods over that period of 13 yr made the 1936 report obsolete. The Committee also took cognizance of the overlap between four major technical organizations, American Society of Civil Engineers, American Congress on Surveying and Mapping, American Geophysical Union, and American Society of Photogrammetry and recommended certain organizational changes in the Surveying and Mapping Division. No doubt these recommendations were taken into account when changes to the organization were being considered in the 1950's.

THE 1950'S

In the early 1950's the Committee on Land Surveys and Titles just about abandoned its hope of straightening out the title situation, largely because the American Bar Association did not appoint a section of a Joint Committee to study the problem. The Committee on Highway and Bridge Surveys was reorganized and substantial work was undertaken on a manual. Efforts toward this objective were spurred on by the advent of the Interstate Highway program.

The Division focused considerable attention on the field of professional development during the 50's. In this connection, the background of the Task Com-

mittee on Status of Surveying and Mapping has been quite completely covered in recent literature; hence, it is not repeated here.

As previously indicated, the Task Committee's studies and recommendations left this Division a legacy of hard work - a rewarding legacy, worthy of the best efforts of the Division. Some of this work was started in the late 1950's and considerable progress was made. The technical committee structure of the Division was changed to conform to the 4 broad fields of surveying which the Task Committee identified and clearly defined in its report. Other committees, of a non-technical nature, on Session Programs and Publications, were created or strengthened to provide a medium for keeping the membership better informed through technical sessions and the publication of papers. A Committee on Research and Development was formed to further progress in this critical field.

THE FUTURE

The Division has a "new look" for the 1960's which has been acquired after a thorough study of the developments of past years. It has four technical committees with broad objectives compatible with the modern concept of surveying and mapping, and three administrative committees, including Research and Development, with heavy and far-reaching responsibilities. The future program includes:

1. Continuous attention to research in surveying techniques and instrumentation.
2. Efforts to broaden the concept of surveying and mapping to encompass the tremendous expansion that is taking place in the scientific fields related to space exploration and oceanography.
3. Assistance and encouragement to appropriate committees of the Society in aiding educational institutions to shape appropriate courses of training in surveying and mapping in accordance with a changing concept.
4. Improvement in communications, through more publications, newsletters, and so forth.
5. Preparation and circulation of abstracts of technical writings on subjects in surveying and mapping.
6. The development of ways and means of emphasizing the importance of the professional attitude and the virtue of true conformance to the Society's Code of Ethics.

These are but a few of the many matters of concern to the 2300 or more members of the Society presently (1961) enrolled in the Surveying and Mapping Division.

ACKNOWLEDGMENT

This outline history of the Surveying and Mapping Division was prepared by Earle J. Fennell, F. ASCE, during his term as chairman of the Executive Committee of the Division (1959-1960). The minutes of the Division meeting of March 23-25, 1960, state: "The paper is a valuable compilation on material from the files of the Division." It was presented by Fennell to a technical session of the Division during the March 1960 ASCE Convention in New Orleans, La.

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MEASUREMENT OF DISPLACEMENT AND DEFORMATION BY GEODETIC METHODS

By F. Kobold[1]

SYNOPSIS

Many methods of determining terrain movements and deformations in structures have been used. However, geodetic methods provide the only means of determining absolute displacements.

All geodetic methods are based upon the measurement of angles or distances, or both. Triangulation involves measurement of angles; trilateration, measurement of distances; and traversing, measurement of angles and distances. Trilateration has not been used in this work, so far as the writer knows, but the development of electronic distance measuring devices may make this a suitable method in the future.

The precision required in this work varies considerably. Mean errors in location may vary from a few centimeters to a few tenths of a millimeter, depending upon the magnitude of the displacement. Measurement of displacements in structures must usually be very precise, as the results are used to verify the hypotheses used in the design of the structure.

It was necessary to provide new equipment in order to meet the extremely high precision usually required in this work. A basic principle in the development of this equipment was to use existing equipment as far as possible, making it suitable for specific purposes by means of inexpensive auxiliary

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equipment. Moreover, the auxiliary equipment was to be so designed that it could be used for various purposes rather than a single, specific use.

Various applications of geodetic methods to terrain movement and deformations in structures are described and specific examples given. The new equipment is described and shown in operation.

PURPOSE OF DEFORMATION MEASUREMENTS

The location of points on the surface of the earth, both horizontally and vertically, is subject to constant change. As a rule, there is no need to measure such changes and, consequently, displacement and deformation measurements are seldom made. However, such measurements may be required to determine the movement of a mountainside that might develop into a landslide and endanger a village unless safeguards were taken. Or it might be necessary to determine deformations in structures such as bridges, tunnels, and dams. Deformation measurements on structures are particularly useful in checking the reliability of assumptions relating to the elastic behavior of the structure. In most cases these measurements are extended to the terrain surrounding the structure. In the case of a dam, for instance, we are not interested only in the movements of the dam; we want to determine whether there are movements, both horizontally and vertically, of the subsoil in the immediate vicinity of the dam due to variations in the level of the impounded water.

The selection of the method of measurement depends upon the accuracy desired and this, in turn, is determined by the purpose of the measurements and by the magnitude of the deformations or movements. To measure terrain movements, for instance, in an area of potential landslides, where the annual velocity has a magnitude of several inches, mean errors of a few centimeters are acceptable. If, on the other hand, the displacement of points on a concrete dam that move a few centimeters is to be measured, the measurements must be such that the mean error does not exceed a few tenths of a millimeter.

Of the many possible methods of determining displacements and deformations, only those shall be discussed that involve geodetic principles. Methods using geodetic principles are disadvantageous in that they almost always require considerably more work than methods using pendulums, clinometers, and other instruments. On the other hand, only geodetic measurements will determine absolute displacements and only by geodetic methods can the directions and magnitudes of the displacements be related to fixed points outside the area of movement.

All geodetic methods are based upon the measurement of angles or distances or both. If only angles are measured, we speak of triangulation. If only distances are measured, we speak of trilateration, a method that is gaining an ever-increasing importance in Geodesy due to the recent development of electronic distance measuring instruments. If both angles and distances between successive points are measured, we speak of traversing. The method of trilateration has probably not been used for the measurement

of terrain movement or deformations up to the present (1958). Both triangulation and traversing have been used for this work.

TRIANGULATION

When triangulation is used to determine displacements, location is usually determined by resection (Fig. 1). From two or preferably three fixed observation points marked I, II, and III, the directions to the points to be located are measured during a given period or, perhaps, during each of several periods spaced over many years. For clarity, Fig. 1 shows only one point, marked 1, to be located. As backsights for the orientation of the directions, distant, well defined points (A, B, C), are used.

The practical solution of this theoretically very simple problem is subject to many difficulties. The major difficulty, which is the only one that will be treated herein, is that, in most cases, the observation points also change location, suffering small displacements in the course of time. It is necessary, therefore, to determine the exact location of station points I, II, III, and so on, and the exact orientation of the reference directions at the time each individual measurement is taken. From the beginning, the station points must be related to fixed points and, at every measurement, this relationship must be checked. It is often very difficult to find points that are not too far away from the landslide or from any particular structure that do not change location over a period of several years.

When the deformations of dams are to be measured, the observation stations are, almost without exception, solid concrete pillars. It is desirable to install reference marks (II_1, II_2, II_3, II_4), in the vicinity of the pillar in sound rock and to measure the directions to these points during each measuring cycle. If sets of directions to these reference marks measured at different times do not agree, the discrepancies indicate that movement of the pillar and reference marks has taken place. However, these movements are related only to the immediate surroundings of the pillars. In order to determine whether the immediate surroundings are subject to displacement relative to more remote surroundings, additional "safety" pillars (IV and V in Fig. 2), are installed. If all possible directions are measured to the various pillars, a complete net is obtained and it can be adjusted by the method of Least Squares. In this manner, the exact location of the observation pillars can be obtained relative to the safety pillars. The orientation of the directions to the various points on the dam is obtained at the same time.

The distance between safety and observation pillar is, generally, about 200 m (660 ft). Because the mean error of an observed direction cannot be reduced below $\pm$ 2cc (0.6 in.), the mean error in position of the observation pillar with respect to the safety pillars amounts to roughly 1 mm. If, in the evaluation, a displacement of less than, say, 1 mm is found, the results cannot be considered to be guaranteed. Then the position of the pillar should be determined relative to the reference marks. The mean error in position should be of the magnitude of a few tenths of a millimeter. If, in the evaluation, it is found that the displacement exceeds about 1 mm, the reference points cannot be used. Under such circumstances, the accuracy of location of the pillar is about one third of what it would have been if the reference points could have been used. Because the distances from the observation pillars to the dam will be approxi-

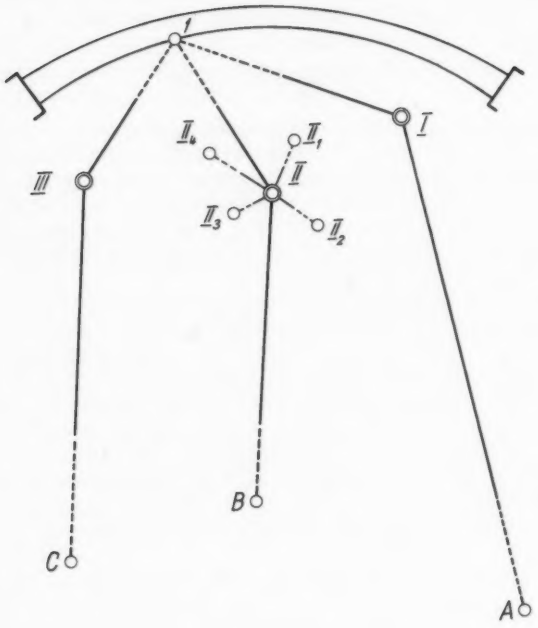

FIG. 1.—DISPLACEMENT MEASUREMENTS BY TRIANGULATION (RESECTION)

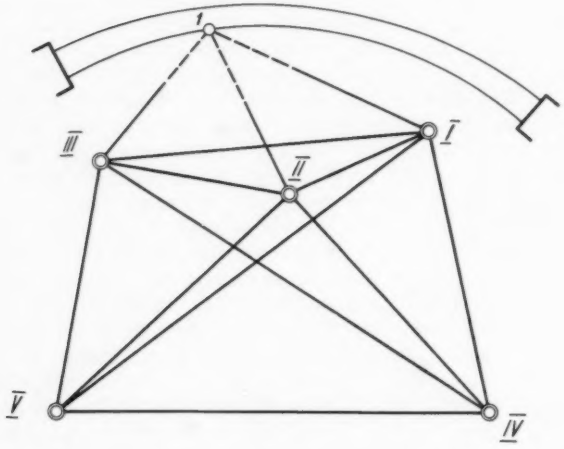

FIG. 2.—SIMPLE FORM OF A PILLAR NET WITH THREE FORWARD (OBSERVATION) AND TWO REAR (SAFETY) PILLARS

mately 200 m, the mean error in position of points on the dam should amount to a little less than 1 mm, if the pillars are located by observations on the reference points; it will amount to as much as 1.5 mm, if the safety pillars are used for location of the observation pillars.

The "creeping" of the pillars referred to previously can be due to various causes. Only two of the causes shall be discussed, namely, creeping over an extended period of time due to variations in water level and creeping within a short period of time due to temperature stresses in the pillar itself.

When creeping takes places over an extended period of time due to the raising and lowering of the water level behind the dam, a part of the forward movement is usually compensated by backward movement. Whereas measured pillar displacement in Switzerland has seldom exceeded a centimeter, instances are known in other countries of pillar displacements over a period of years of several centimeters. Fig. 3 shows displacements of an observation pillar and probably of the rock in the vicinity of the dam, over an extended period. The downstream (forward) movement of pillar A amounted to 14 mm over a period of 5 yr. The relationship between displacement and water level is quite obvious. While the water level was rising in the period from A to C, the displacement was downstream. Lowering of the water level in the period from C to D, resulted in an upstream displacement. This phenomenon repeats itself over the entire period of measurement.

The measurement of displacements in the position of the pillars related to the level of the lake is not as difficult as the determination of changes in the position of the surface of the pillars over short periods of time. The displacements of these surfaces, upon which the instrument or targets are set up, relative to the ground in which they are set, are due almost always to unequal heating or cooling of the pillars. Generally, these displacements are not due to actual changes in the position of the entire pillar but to rotation of the pillar with respect to its footing; there is no displacement of the bed rock or the immediate surroundings. As shown in Fig. 4, measurements made on a sunny day in October indicated that the surface of the pillar was displaced with respect to the ground by 1.0 mm, in the course of 2 hr. The position of the surface of the observation pillar was determined at 8:40 and 11:45 in the morning, and at 2:00, 3:30, 3:50, and 4:00 in the afternoon. The precision shown by the mean error ellipse of the first determination was the same for all observations. The creeping is real. It takes place along a straight line. As soon as the temperature difference between the side exposed to the sun and the shady side exceeds a certain amount, in this case after 11:45 a.m., the motion begins. It is reversed when the temperature difference falls below some amount. These movements are not proportional to elapsed time and they are probably not at all countinuous but are "jerky" in nature.

The rotation of pillars that takes place while sets of directions are being measured has been known to geodesists for a long time. The displacement of pillars is not as well known because, for normal geodetic work, such displacements have no influence on the results. The situation is completely different when measurements are made to determine displacements on dams. During such measurements every precaution must be taken to prevent unequal heating or cooling of the pillars. These precautions are necessary for both observation and target pillars. The most elaborate precautions known to the writer were taken in Portugal. A rotating cupola with an observation slot was built around each pillar.

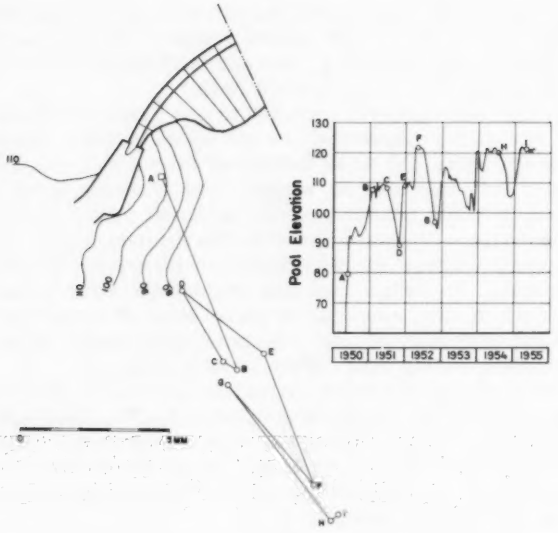

FIG. 3.—LONG PERIOD DISPLACEMENT OF AN OBSERVATION PILLAR IN THE VICINITY OF A DAM IN PORTUGAL

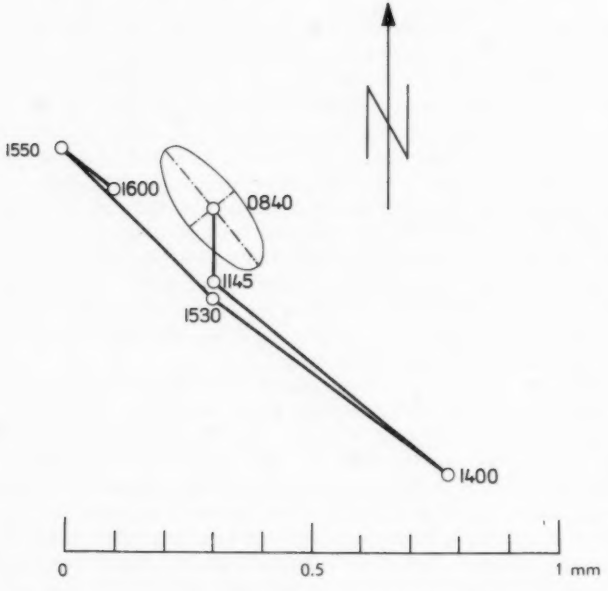

FIG. 4.—SHORT PERIOD DISPLACEMENT OF AN OBSERVATION PILLAR

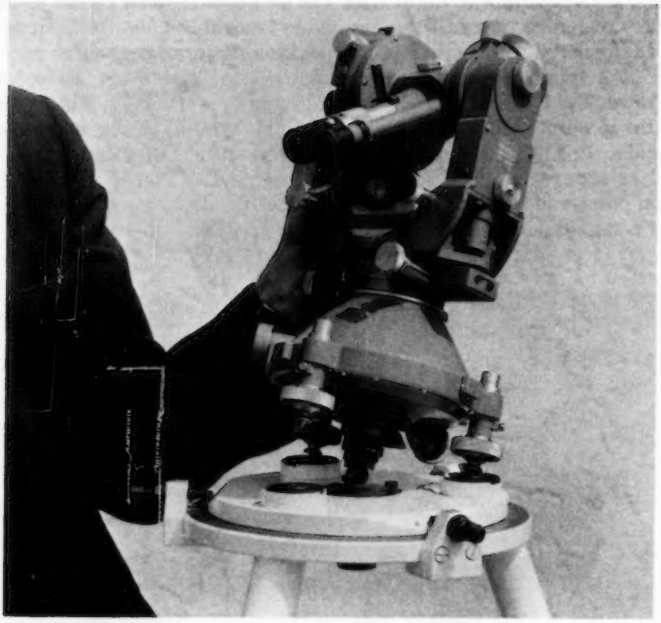

FIG. 5.—CENTERING OF WILD-HEERBRUGG T3 PRECISION THEODOLITE ON A PILLAR

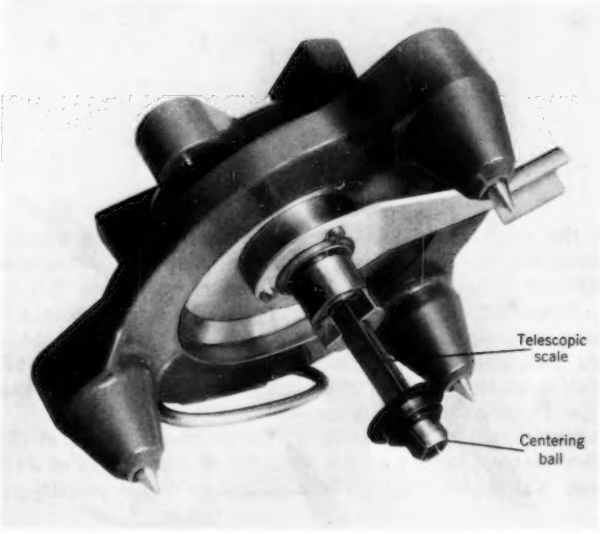

FIG. 6.—TRIVET FOR KERN-AARAU THEODOLITES AND TARGETS SEEN FROM BELOW

Because it is possible to determine the location of observation pillars with an accuracy of tenths of a millimeter under favorable conditions, it is necessary to equip theodolites with special devices that can guarantee a centering accuracy of a tenth of a millimeter. Figs. 5 through 9 show the accessories manufactured by Kern and Wild for their instruments. In Fig. 6 the trivet rests on three points. It is centered with the ball on the lower end of the telescopic scale. In Fig. 7 the upper part is tilted on the ball joint until the bull's-eye level is accurately centered. Then the centering sleeve is directly above the station point and the supporting surface for theodolite or target is horizontal within a minute.

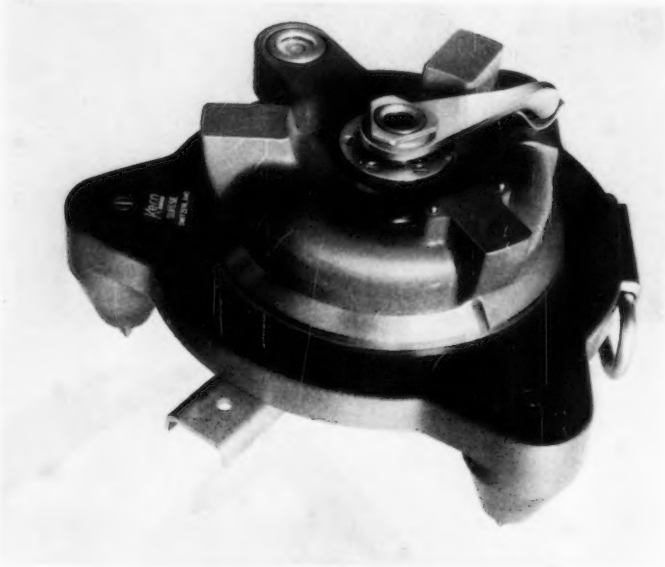

FIG. 7.—TRIVET SEEN FROM ABOVE

Basically the same problems are encountered on dams and in landslide areas in determining the momentary position of observation stations. Usually it is very difficult to find points in the vicinity of landslides that do not vary in location. Frequently it is necessary to tie in remote points, located on mountain peaks or ridge lines, which, experience has shown, have smaller displacements than points located on slopes of the valleys. The accuracy of the determination of the position of station points naturally decreases as the distances to the reference points increase.

A few examples will serve to clarify the problem. The first (Fig. 10), is concerned with measurements in the vicinity of the village of Peiden in the lower Lungnez Valley. In Fig. 10 station points in the vicinity of Pitasch-

FIG. 8.—DKM3 PRECISION THEODOLITE ON A TRIVET

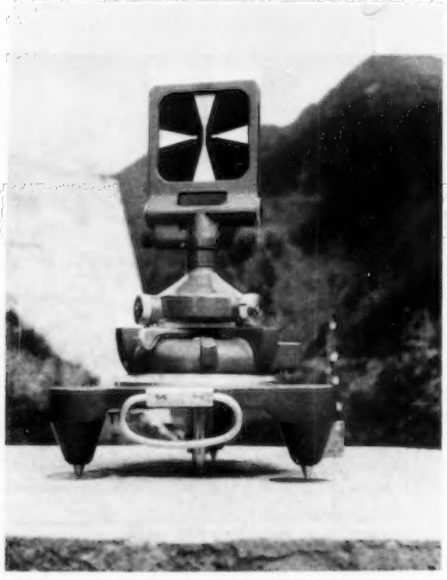

FIG. 9.—TARGET AND TRIVET

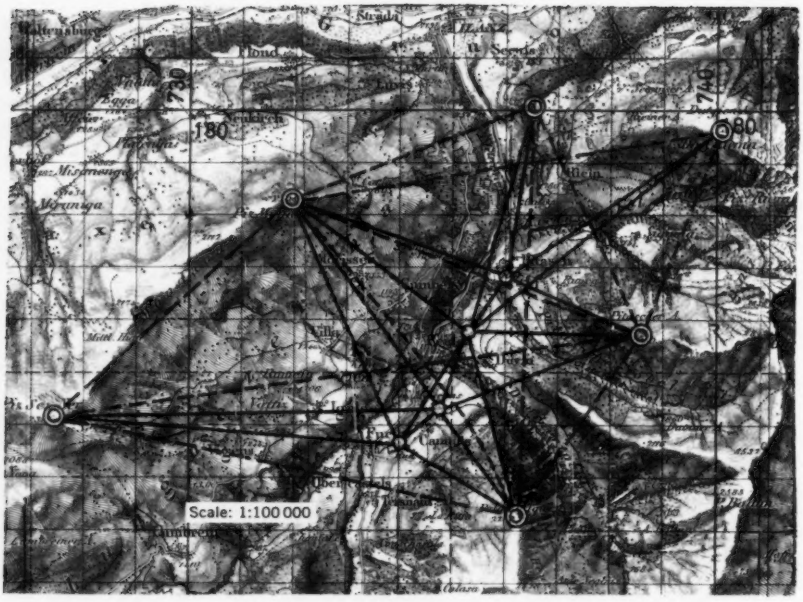

FIG. 10.—DISPLACEMENT MEASUREMENT IN LOWER LUNGNEZ

FIG. 11.—DISPLACEMENT MEASUREMENTS NEAR BRAUNWALD

Duvin-Camuns are marked with a single circle. Third order triangulation points are marked with a double circle. In order to examine terrain movement on the left side of the valley in the Peiden-Cumbels area, directions were observed from four stations on the lowest terrace of the right side of the valley. It was questionable whether the four stations as well as the neighboring third order points were fixed in position. To determine displacements, the third order points were adjusted as an independent net and compared with Federal coordinates by means of Helmert transformations. No point showed any appreciable displacement. Then the four stations were located by multi-point resection. A comparison with the former coordinates

FIG. 12.—LANDSLIDE REGION OF SCHUDERS; G MARKS THE GYRENSPITZ

showed that all four stations had moved over a period of 10 yr, from 5 cm to 10 cm, toward the bottom of the valley.

The second example relates to measurements in Braunwald (Fig. 11). It was assumed that points on mountain peaks and ridge lines had not changed position since the fourth order triangulation was run 30 yr ago. Location and elevation nets were run to connect these points (marked with two concentric circles in Fig. 11). Each net was adjusted independently. A comparison with Federal coordinates was made by means of Helmert transformations. Taking into account the mean errors of the old and new measurements, as well as the fact that the two determinations were made in completely different ways, no

displacement was indicated. With reference to these points, some of the points on the slope terraces showed quite considerable movement. Fig. 11 shows settlements.

In the landslide area of Schuders (Fig. 12), it was extremely difficult to find fixed reference points for deformation measurements. In Figs. 12 and 13 the third order triangulation points are marked with a triple circle. The area marked in Fig. 13 was examined for terrain movement and a rather dense triangulation net was established for the examination. Only the third order points, indicated by two concentric circles, could be assumed as fixed. It was known that the third order point "Schuders" (three concentric circles), did not maintain a fixed position. However, it was necessary to use the point because without it a net could not be established. Because it could be located easily by resection from the fixed points, the procedure adopted was to compute its coordinates at the beginning and at the end of the measurements and to interpolate between these values to obtain its location at a given time of observation. Fig. 14 indicates that this procedure was reasonably correct. Displacement amounted to 12 cm over a period of four weeks. The mean error ellipses indicate that it is a real displacement. Interpolation of position for a given time can be considered reliable because the check sight of September 6 is very close to the interpolated location and, furthermore, the interpolated location of September 15 lies within the mean error ellipse of that date, although, admittedly, it is a larger ellipse.

For all all these terrain measurements, changes in elevation as well as in position are required. When triangulation methods are used, whether it be for the determination of slide movements or of deformations in structures, vertical angles are measured as well as directions. Here again the problem of finding points that do not vary in elevation is encountered. It is usually necessary to utilize points that are distant from the structure or area in which the measurements are to be made. The precision with which elevations can be determined depends on the precision with which the vertical angles are measured and the length of the sights.

When measuring terrain movements, the mean errors of elevation are about those encountered in conventional triangulation. A much higher precision is obtained when determining the elevations of observation pillars used for the determination of deformations in dams. In the net connecting the pillars all vertical angles are measured so the mean errors lie in the neighborhood of $\pm 2^{cc}$ (0.6 in.). Because of the short sights, a remarkably high precision is obtained for elevations. After the adjustment of the net, it is possible to reduce the mean errors of the elevations of the observation pillars to 0.3 mm. This precision corresponds to that of good precise leveling. The precision of the elevations of the plugs in the dam is not as good. Mean errors in the elevations of these plugs can be as much as 1 mm, because very frequently the sights from the pillars to the plugs are disturbed by abnormal refraction.

TRAVERSING

The use of triangulation for the determination of terrain movements and deformations in structures has several disadvantages. To begin with, the work - observations and computations - may extend over a period of days or even weeks. Furthermore, the precision of the method is limited; some data

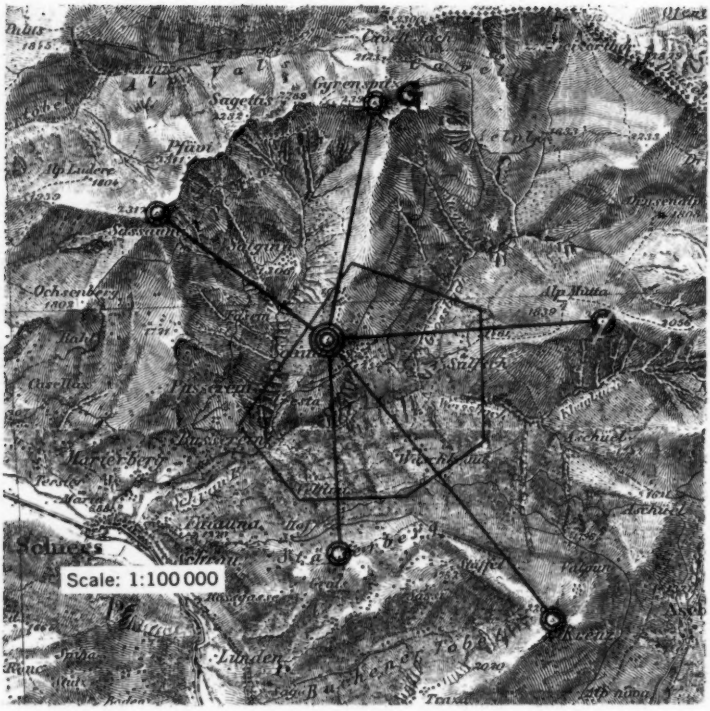

FIG. 13.—DISPLACEMENT MEASUREMENT IN SCHUDERS

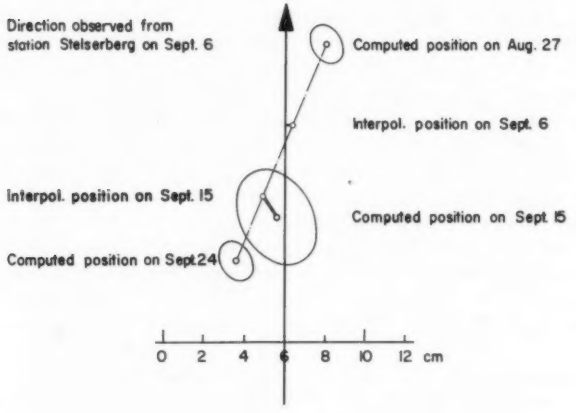
Direction observed from station Stelserberg on Sept. 6
Computed position on Aug. 27
Interpol. position on Sept. 6
Interpol. position on Sept. 15
Computed position on Sept. 15
Computed position on Sept.24
0 2 4 6 8 10 12 cm

FIG. 14.—DISPLACEMENT OF THIRD ORDER POINT ABOVE SCHUDERS

have already been given. If information is required on the magnitude of the displacements of a few points that are not in close proximity, the mean errors to be expected using triangulation methods will generally be satisfactory. This is not so when displacements of adjacent points must be determined. With triangulation methods, all points are located independently. The displacement of points in a landslide area which are 100 m (330 ft) apart, for example, can be in error by several centimeters, whereas a direct measurement would have a mean error of 1 cm.

From this it is obvious that when the distances between the points to be located can be measured directly, traverse will provide a higher precision than triangulation. In any event, traversing is always preferable when direct distance measurements are practicable. For example, to determine terrain movements with a position accuracy of ± 1 cm, a traverse using conventional surveying methods is satisfactory and far simpler than triangulation which would not provide as high a precision. Of course, it is necessary to determine the exact location of the initial and final point of the traverse. This can be done by tying these points in with fixed points which are not subject to movement.

Where greater accuracy is required such as, for instance, in mean errors of less than 5 mm, with courses about 50 m in length, the measurement of both distances and angles requires particular care. For the measurement of distance, particular care must be devoted to the proper calibration of the measuring device. The precise testing of subtense bars and other measuring devices such as bars, rods, wires, and tapes cannot be done in a simple manner. The measurement of the angles presents difficulties because a centering accuracy of roughly 1 mm is required. Such accuracy cannot be achieved with a plumb bob. It is necessary to use a carefully adjusted optical plummet or a centering rod equipped with a good bull's-eye level. Centering tripods facilitate the work.

Such traverses are well suited to the investigation of terrain movement if the movement is to be analyzed by studying the displacement of a large number of closely spaced points. Usually these points lie roughly in a horizontal plane or on lines of steepest gradient. Fig. 15 shows an example of such a measurement made in the landslide area of the village of Peiden. Peiden is in the center of the figure and Peidenbad at the right (compare with Fig. 10). The four observation stations on the right side of the valley were used as fixed points to determine the coordinates of triangulation points in the landslide area by forward and combined resection. Traverses with an average course length of 50 m were used to connect these triangulation points. The mean error of location of adjacent points was less than 4 mm. Distances were measured with a 2-m invar subtense bar. The centering tripod was used for centering. Tying in the end points of the traverses by triangulation was somewhat questionable because the location precision obtained by triangulation was no greater than that obtained by traverse. Even so, the closure errors were less than 2 cm throughout. The points were tied in by one of the conventional methods.

Traversing this way - position accuracy of adjacent points of 3 mm to 5 mm - cannot be considered to be precise traversing. A position accuracy of a few tenths of a millimeter for contiguous points is required in high precision traversing. Then the traverse courses are seldom longer than about 24 m, and may be as short as 10 m, when the traverse runs through the control

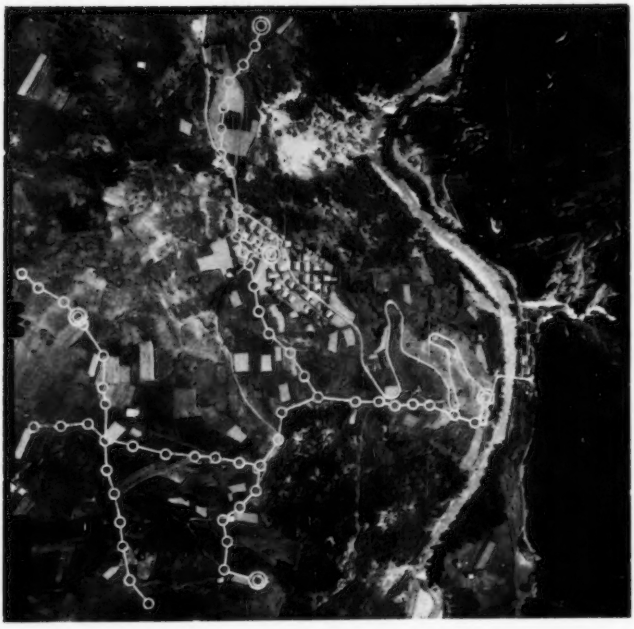

FIG. 15.—DISPLACEMENT MEASUREMENTS IN THE LOWER LUNGNEZ VALLEY BY TRAVERSING

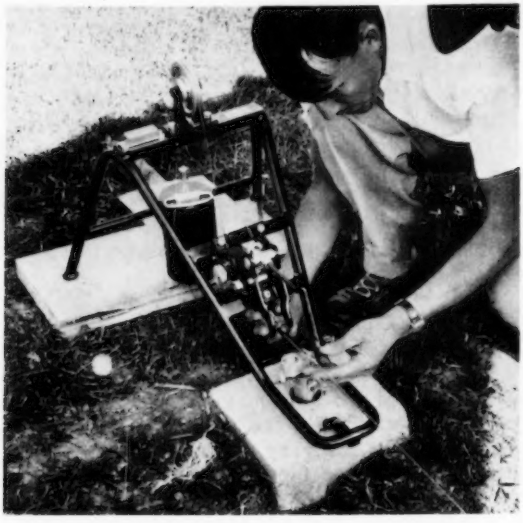

FIG. 16.—TENSION FRAME FOR INVAR WIRE EQUIPMENT BEING USED ON LANDSLIDE MEASUREMENTS IN OPEN TERRAIN

gallery of a dam. Both the measurement of distance with a mean error of the order of magnitude of tenths of a millimeter and the measurement of the traverse angles by a theodolite with a mean centering error of not more than a tenth of a millimeter, require special instrumentation. There must be exact duplication of centering over a traverse station by theodolite and target.

Special instruments that could provide such high precision were not available until recently; they had to be created. The fundamental concepts for these instruments stem in large part from the work of the Geodetic Institute of the Swiss Federal Institute of Technology; Kern and Co., Aarau, designed and manufactured them. A basic principle in the development was to use existing instruments to as great an extent as possible, converting them for specific uses with inexpensive auxiliary equipment. Moreover, the auxiliary equipment was so designed that it could be used for various purposes rather than a single, specific use. (W. K. Bachmann, Lausanne, Switzerland, has developed designs for equipment with the same purpose that differ from the equipment described herein.)

For the determination of distance with very high precision requirements, measurement with invar wire or invar tape, as it has been used by the surveyor for base line measurement for over 50 yr, is the only method to be considered. It is apparent from the abundant literature on the theory and practice of invar wire measurement, that the actual precision is substantially less than that which the intrinsic agreement of the observations would lead one to believe. Systematic errors that are difficult to comprehend, assert themselves in disagreeable ways. The application of the tension, the free motion of the sheaves, the setting up of the wire in alinement and elevation on the measuring section with the help of the small scales, and the correction for sag are all critical. Particularly bothersome are the cracks that occur from time to time in the wire or tape.

For these reasons it has been customary for a long time to use more than one set of equipment in making measurements. Three wires are used on all deformation measurements on structures just as is the case on base line measurement. This precautionary action is found to be justified again and again. Observation of deformations in dams and tunnels has repeatedly shown that, during the course of the measurements, the length of one of the wires will suddenly change by several tenths of a millimeter. Such changes can be determined by a comparison of the results of the three wires and, after introducing appropriate corrections, measurements made with the changed wire can still be used. To determine the correction, the wires should be compared be measuring a test length in both directions with all wires. In addition, the lengths of the wires should be checked from time to time by comparison with standard wires.

From the consistency of repeated measurements made with all wires, a mean error that averages less than ± 0.1 mm can be computed. On the basis of information in the literature as well as the writer's experience since the spring of 1954, however, the mean error for traverse lengths of from 10 m to 20 m measured with three wires, may be assumed to be ± 0.15 mm on the average. This value is confirmed by the closure error of traverses.

The equipment developed for very precise measurement of distance includes, in addition to a wire or tape of invar, a tension frame with hanging weight, sheaves or pulleys for the wire connecting weight and tape, and an index holder. It is suitable for distance measurements between cemented pins

FIG. 17.—TENSION FRAME FOR INVAR WIRE EQUIPMENT ARRANGED FOR THE MEASUREMENT OF COURSES ALONG RAILROAD RAILS

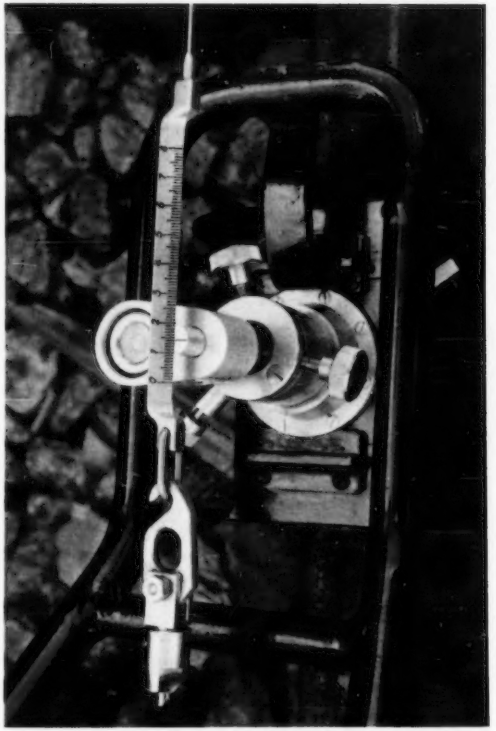

FIG. 18.—VERTICAL INDEX HOLDER, REFERENCE MARK AND SCALE ON THE INVAR WIRE, READY FOR READING

in arbitrary locations or between small conical holes in railroad rails. Fig. 16 shows the tension frame with hanging weight and connecting wire, arranged to measure the distance between metal pins cemented into deep pedestals. The reference mark is set in the small, conical hole in the pin of the pedestal (at the right in the picture). The invar wire terminates in a short, metal scale. Tension is applied by a hanging weight of 10 kg, that is connected to the invar wire by means of an ordinary wire that passes over two pulleys. Adjusting fixtures on the tension frame allow the metal scale to be moved laterally and vertically to position it correctly. Fig. 17 shows the tension frame set up for a measurement along a railroad rail. In Fig. 18, the index holder with the reference mark and the bull's-eye level, as well as the metal scale on the invar wire, are clearly shown. Fig. 19 shows the measurement of distance between pins in pillars. Note the use of the pillar ground plate, previously mentioned, for the positioning of the reference mark over the pin.

The problem of centering a theodolite set up on a tripod, with an accuracy of a tenth of a millimeter, is easily solved with the double circle theodolite, because its alidade moves on a ball bearing and no mechanical axis of rotation exists. Therefore, very accurate centering with the optical plummet, which is built into the alidade, is possible. Observations in diametrically opposite positions of the alidate eliminate entirely angle and eccentricity errors of the optical plummet. A ball joint centering tripod with special arrangement for very accurate centering to a tenth of a millimeter is shown in Fig. 20. A similar adaption exists for the tilting-plate tripod. Centering is performed in two steps. First, preliminary centering is accomplished by plumbing the centering rod of the tripod using the bull's-eye level attached to the rod. The mean error of this centering amounts to somewhat less than ± 1.0 mm. Then precise centering follows, utilizing the optical plummet and the alidade level. With a leveling accuracy of a half division, that corresponds to about 10 in. at an instrument height of 1.4 m, a centering accuracy of less than 0.1 mm is obtained. Thus, the high accuracy of centering required for precise traversing is attained. To check this centering, a test traverse should be run. Anyone doing it for the first time will devote considerable time to the centering. This time is well spent because, after a little practice, the average observer will find that he can perform the centering in less than 10 min. In any event, it is possible to measure 40 traverse angles in less than a day, as was demonstrated in the case of one of the dams on which deformation measurements were taken. Good organization of the survey party is a decisive factor; the theodolite and targets must always be interchanged.

The centering accuracy here computed has been verified by various test measurements made at the Geodetic Institute. However, important corroboration has resulted from triangulation closure and traverse closure errors on structures.

Using the invar wire and the special centering, traverse measurements can be relied upon to produce mean errors in the sides of about ± 0.15 mm and in the angles of $\pm 10^{cc}$ (3.2 in.). Therefore, the central point of a traverse 400 m long with sides of 11 m, has a mean error in the direction of the traverse of ± 0.5 mm, and transversely, of ± 2.0 mm. The mean error of the total length amounts to a fraction of a millimeter. For such traverses, however, it is of the greatest importance that adjacent points be located with respect to one another with an accuracy of a few tenths of a millimeter.

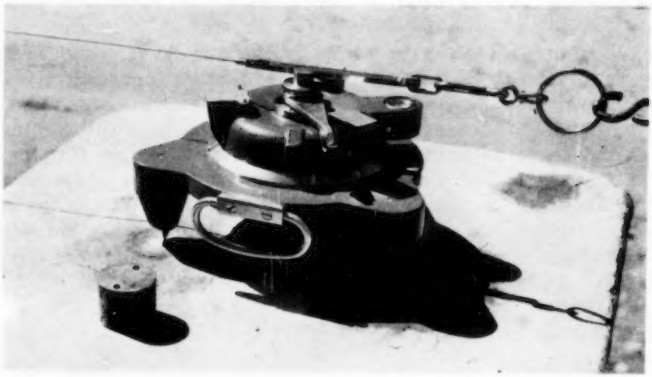

FIG. 19.—WIRE MEASUREMENT ON A PILLAR USING A TRIVET

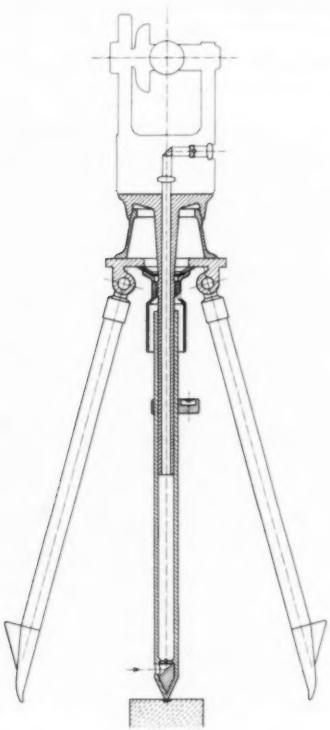

FIG. 20.—BALL JOINT CENTERING TRIPOD WITH SPECIAL ARRANGEMENT FOR VERY ACCURATE CENTERING TO A TENTH OF A MILLIMETER

Two examples will be used to demonstrate applications of precise traversing.

The first example relates to the Sambuco Dam. As is shown in Fig. 21, a traverse was laid out in the uppermost control gallery, only a few meters below the crown. The lengths of the traverse sides were so chosen that two traverse points were located on each block of concrete at this lift. In order to connect the traverse with the pillar network, the points defining the traverse line at each end of the traverse were transferred to the crown of the dam. The offsetting and plumbing equipment used was sufficiently precise to maintain a positioning accuracy of tenths of a millimeter. Measuring sections

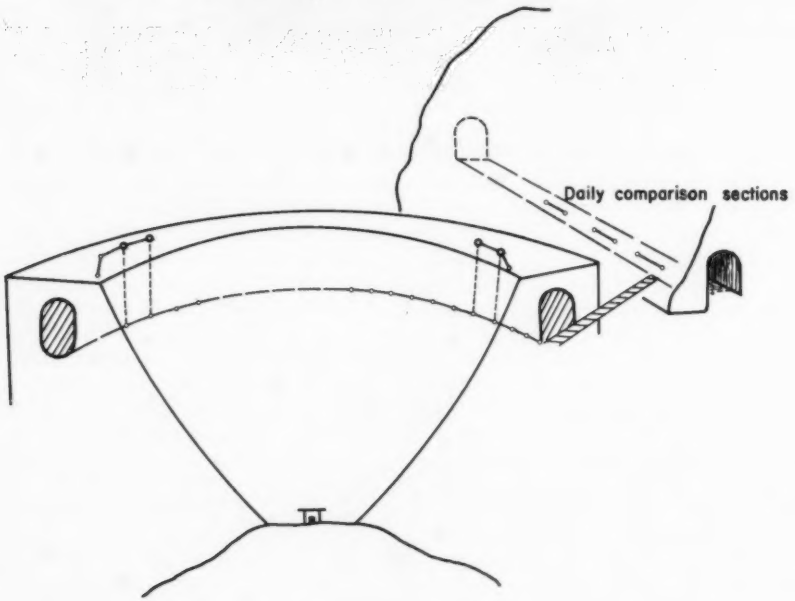

FIG. 21.—SCHEMATIC REPRESENTATION OF THE TRAVERSE IN THE TOP GALLERY OF THE SAMBUCO DAM

were installed in the adjacent road tunnel for the comparison of the measuring wires. Fig. 22 shows the construction of the traverse points used in the control gallery. Each point consists of a brass pin with a small conical hole. Also shown are the index carrier with its conical point, the bull's-eye level, and the measuring mark, as well as the cylinder with screws and springs used to plumb the index carrier. Fig. 23 shows a survey party measuring a traverse angle. Fig. 24 shows a target set up on a tripod. A distance measure made with an invar wire is shown in Fig. 25. Note the two tension frames and the invar wire.

The results of traverse measurements made in the spring of 1956, the fall of 1956, and the spring of 1957 are shown graphically in Fig. 26. The

FIG. 22.—TRAVERSE POINT AS USED IN THE CONTROL GALLERY

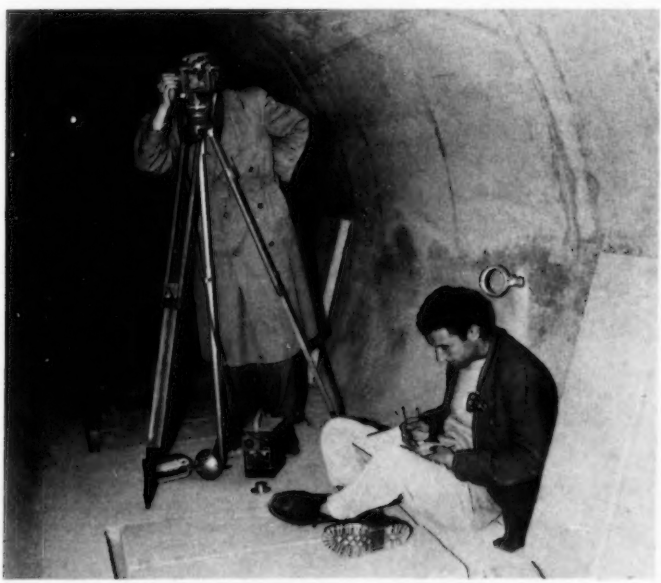

FIG. 23.—TRAVERSE ANGLE MEASUREMENT, USING SPECIAL CENTERING TRIPOD

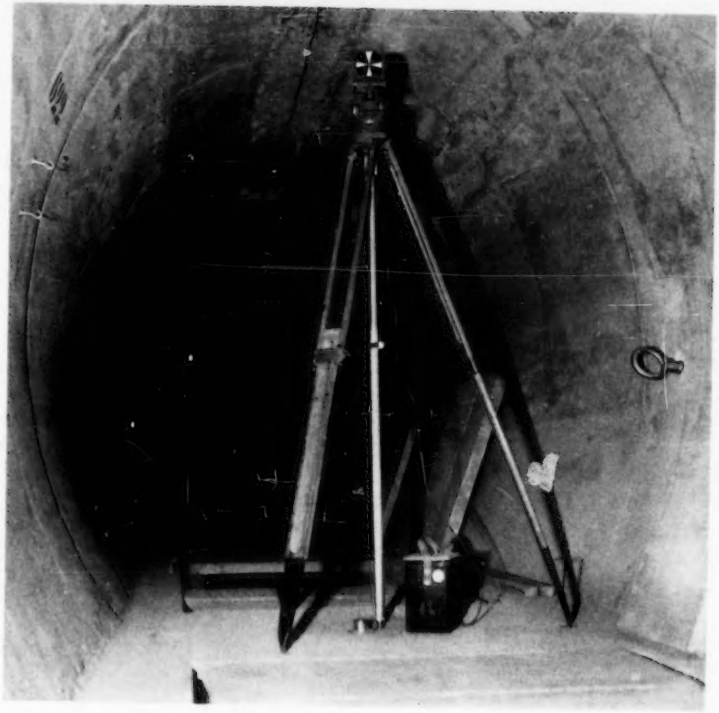

FIG. 24.—TARGET ON SPECIAL CENTERING TRIPOD

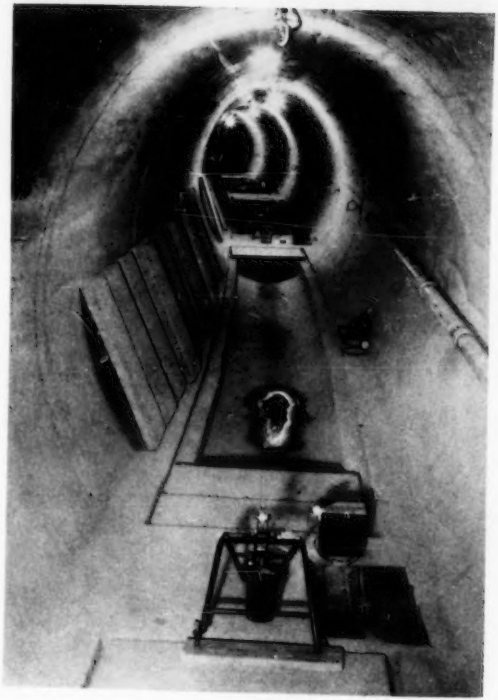

FIG. 25.—DISTANCE MEASUREMENT IN THE TOP CONTROL GALLERY

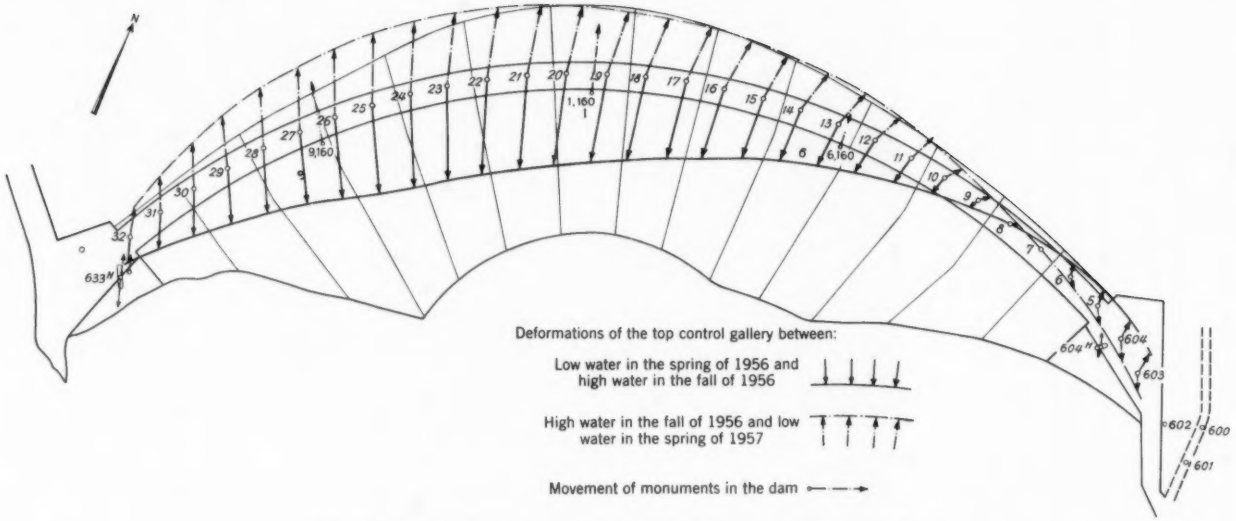

FIG. 26.—HORIZONTAL DEFORMATIONS OF THE SAMBUCO DAM

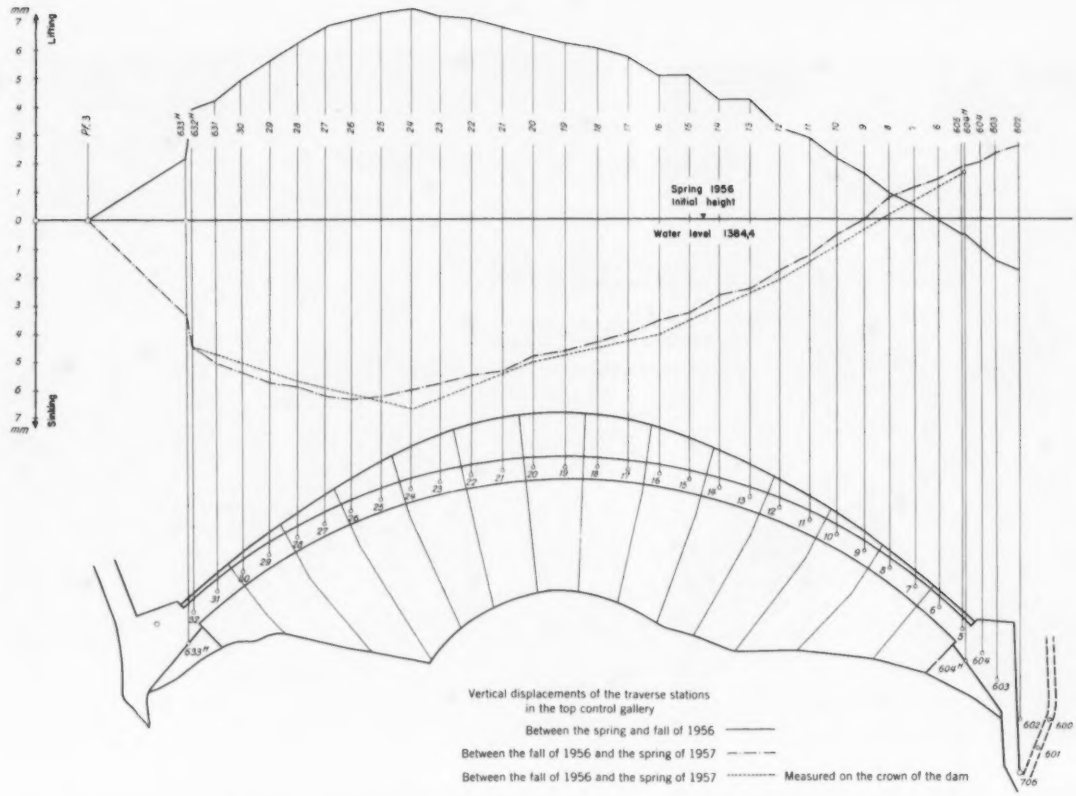

FIG. 27.—VERTICAL DISPLACEMENTS OF THE SAMBUCO DAM

scales used are: plan, 1:2000; displacement of traverse stations, 1:2; mean error ellipse of connecting points 633H and 604H, 1:2, computed from the closure on the pillar net. It is apparent that the forward motion of the crown with a full reservoir corresponds approximately to the backward motion with an empty reservoir. It is also apparent that movement is not symmetrical about the midpoint of the dam. A fact particularly worthy of note is that the filling of the lake puts the western abutment in motion toward the valley, whereas the eastern abutment moves upstream. These opposed motions indicate that there is a point in the uppermost gallery that does not vary in position as the lake is filled or emptied. This point lies between traverse stations 7 and 8.

FIG. 28.—RAILROAD BRIDGE OVER THE LANDQUART RIVER AT KLOSTERS WITH SUBSEQUENT LOOP TUNNEL

The deformation diagram clearly indicates that the high precision of the traverse is necessary in order to comprehend the unsymmetrical behavior of the dam. Uplift and settlement of the traverse stations are closely related to horizontal displacement. Vertical motion was determined by precise leveling. Fig. 27 shows that the dam in the vicinity of traverse station 24 rises about 7 mm when the lake is filled, and settles about the same amount when the lake is emptied. Whereas the west abutment does not vary in height, the east abutment rises as the lake surface falls and settles as the lake fills. Traverse stations 7 and 8 remain fixed in elevation regardless of the reservoir level. The scales of Fig. 27 are, plan, 1:2000; vertical displacement, 5:1.

The second example relates to deformation measurements on the loop tunnel of the railroad at Klosters. It has been known for a long time that landslide movement is taking place on the Davos side of the Landquart River and that the tunnel is, consequently, endangered. Whether the landslide movement is the same for all sections of the tunnel or the central section of the tunnel moves less than the sections in the vicinity of the portals, can be established only by displacement measurements. It is obvious that such problems can be solved only by running a traverse through the tunnel. The traverse laid out in the loop tunnel had sides 24 m in length.

The end points of the traverse were tied together by triangulation, utilizing triangulation stations on the right bank of the Landquart River, that could be assumed to be free of displacement. Fig. 28 shows the first triangulation scheme which involved the measurement of individual sides using the two-meter invar subtense bar. The connecting net between fixed points on the right bank of the Landquart River and the arrangement of the traverse in the tunnel are shown schematically. The final triangulation scheme, shown in Fig. 29, had a base line between stations 83 and 84 on the railway station grounds, that was measured with invar wires. Because traverse points on the ties or on the rails in the tunnel would be subject to displacements not related to tunnel displacements, it was decided to drive metal spikes in the brick walls of the tunnel (Fig. 30) and locate these spikes by intersection from traverse points. Information on displacement of the points is given in Fig 31. It is obvious that the central section of the tunnel remains fast while the portal sections are in motion.

The use of precise traversing seems to be limited to displacement measurements on structures such as dams, bridges, and tunnels, carried out again and again over a period of years.

However, it is also useful for terrain displacement measurements where earth slides can be detected by very accurate distance measurements with tape or wire. The example to be investigated is a limited area of perhaps 100 m by 100 m, within a larger earth mass, all parts of which have a similar motion. The traverse method can then be used by constructing pedestals in the ground and, from time to time, measuring the distances and angles between these pedestals. An example from the slide zone of Peiden is shown in Fig. 32. Through accurate measurement of distances and elevations it was to be determined whether the movement in the partial area is uniform. Pedestals were arranged in a cross-like pattern with a spacing of 24 m between pedestals. The results of the measurements are shown in Fig. 33. In the vertical leg and the western section of the horizontal crossbar, the displacements are, for the most part, less than 1 mm. The displacements in the eastern section of the horizontal crossbar indicate a definite unbalanced motion although, to be sure, the magnitude of these displacements did not exceed 4 mm over a period of $2\frac{1}{2}$ yr.

It should also be pointed out that deformations on dams can be determined by measuring vertical angles. Usually pendulums are hung in the vertical shafts of dams and displacements determined by measuring the distance from the pendulum wire to the walls of the shaft from time to time. This simple method that yields relative, but none the less, very accurate results, is applicable only to vertical shafts.

In inclined shafts, vertical angles are measured either from a pillar at the base of the shaft or from an observation station on the crown of the dam to

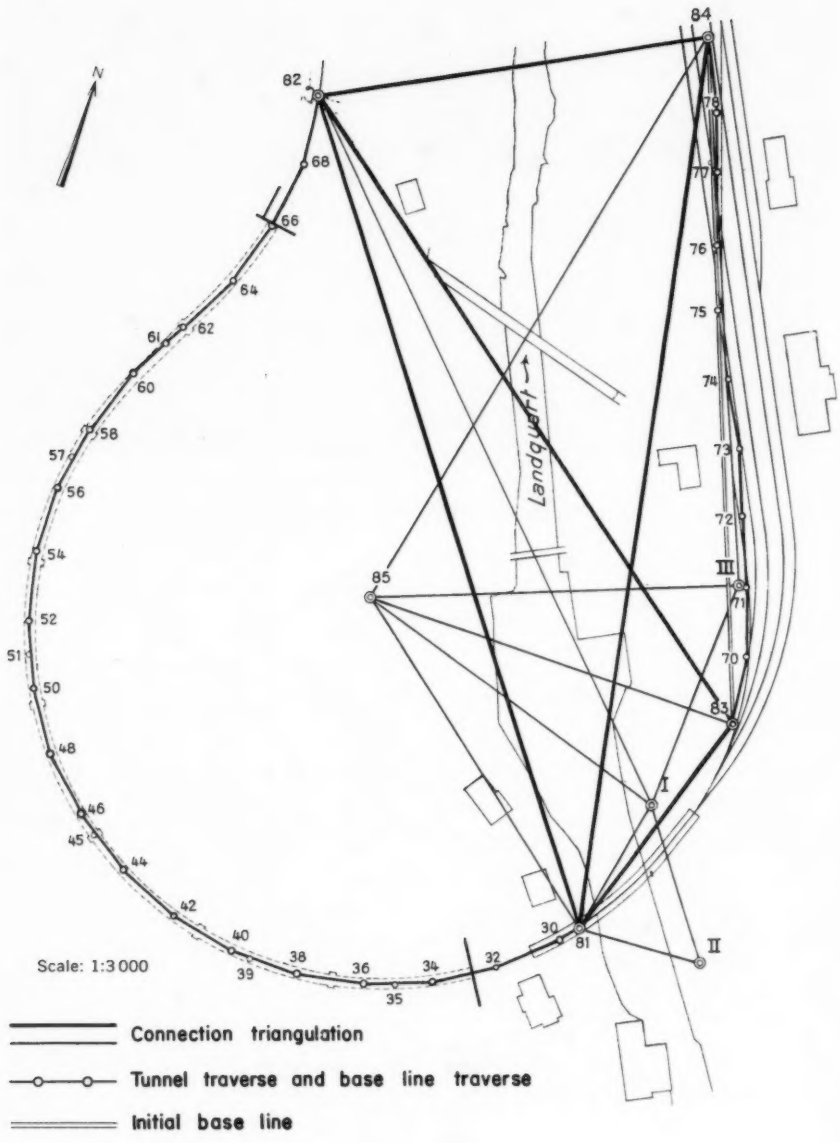

FIG. 29.—KLOSTERS TUNNEL

FIG. 30.—METAL SPIKE IN THE TUNNEL WALL

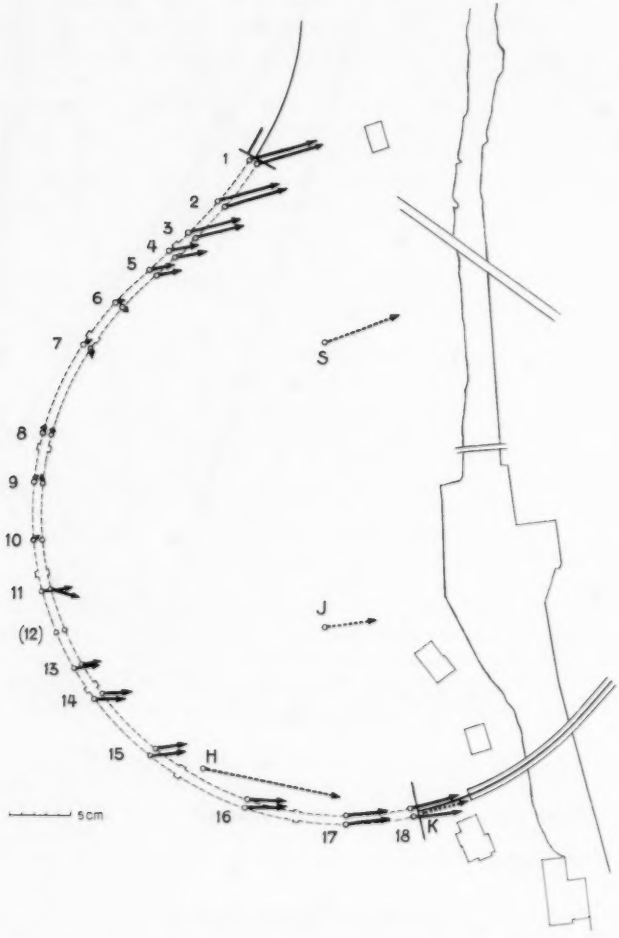

FIG. 31.—DISPLACEMENT OF THE SPIKES BETWEEN 1953 AND 1956

FIG. 32.—SECTION OF THE SLIDE REGION OF PEIDEN

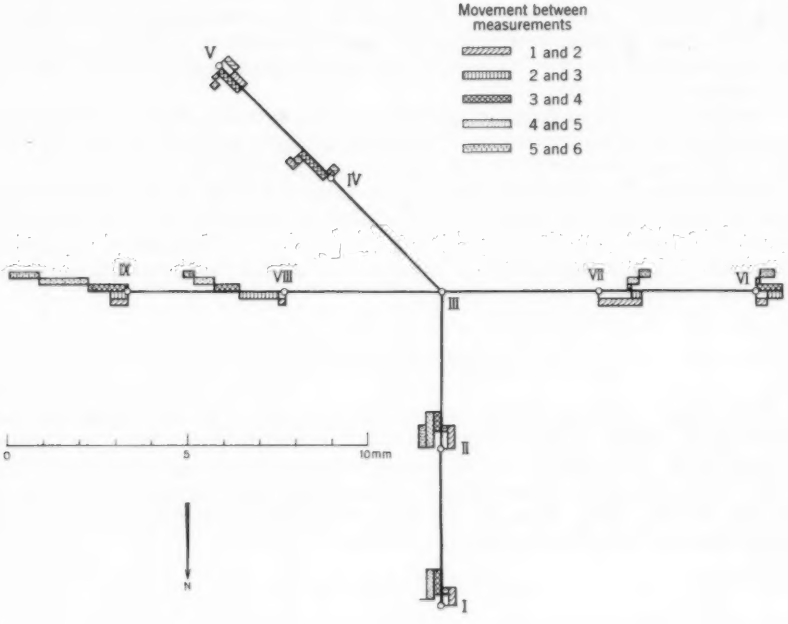

FIG. 33.—MEASURING CROSS IN BARDIALA, RESULTS OF MEASUREMENTS (DISPLACEMENTS) TO A SCALE OF 5:1

targets at various points in the shaft. Because of the steepness of the line of sight, it is necessary to make careful level measurements (of the horizontal axis of the theodolite) on each sight and correct the measured vertical angles accordingly. Accurate observations and acceptable procedure yield deformation curves that are only slightly inferior in precision to those obtained from pendulum observations.

In both the pendulum and the vertical angle method, the fact that the plumb line is not identical for maximum and minimum water levels must be considered. The change in the direction of the plumb line can be determined by troublesome computations based upon the water surface elevations. It should be emphasized that estimation of this change will not yield accurate deformation data or even approximate data. The angle changes are very small. On a high dam and depending upon the shape of the reservoir, the change amounts to a few centesimal seconds at most and is of the order of magnitude of the measuring precision.

CONCLUSIONS

Geodetic methods almost always require considerably more work than other methods, but only geodetic methods can be used to relate the direction and magnitude of displacements to fixed points outside the area of movement. The work involves careful planning, good organization, and special equipment.

Triangulation has several disadvantages, particularly when the displacement of adjacent points must be determined. Traverse will provide a higher precision than triangulation when the distances between points to be located can be measured directly. Traverse is always preferable when direct distance measurements are practicable.

Precise traverse is particularly suited to the determination of deformation measurements on structures such as dams, bridges, and tunnels. Its application to the measurement of terrain movements is more limited.

Measurement of distance with high precision is still made with the invar wire or invar tape. However, special equipment and constant calibration are required to maintain precision at the desired level. To maintain consistent precision in angle measurement, special equipment that provides quick and accurate centering is necessary.

ACKNOWLEDGMENTS

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Journal of the
SURVEYING AND MAPPING DIVISION
Proceedings of the American Society of Civil Engineers

DISCUSSION

Note.—This paper is a part of the copyrighted Journal of the Surveying and Mapping Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. SU 2, July, 1961.

SURVEYS FOR THE LAKE PONTCHARTRAIN BRIDGE[a]

Discussion by Shu-t'ien Li

SHU-T'IEN LI,[2] F. ASCE.—The author has presented a lucid account of the overall survey control for the 23.8-mile Lake Pontchartrain Bridge over open water. The control measures used prior to and during construction achieved the required accuracy in an expedient manner. Even though the author has emphasized only main features of the control without going into refined technical aspects, his paper constitutes a significant account of handling an extremely large location and construction survey under unusual circumstances.

Vertical Control.—This paper is concerned with geodetic control. In his introduction, the author states, "It was decided that the overwater length of this bridge should be a straight line, or great circle, connecting P. I.'s on the north and south shores."

Undoubtedly, the author is aware of both geodetic horizontal and vertical controls. He has, however, presented all main features concerning horizontal control, but has only implied in the preceding quote the vertical control. Some further treatment of this subject would enhance the paper.

In this particular bridge, as Wayne F. Palmer, F. ASCE, of the consulting engineers who designed this bridge, has said, "If the deck were kept level and did not follow the contour of the earth and took off tangent at one shore, it would be 380 ft high at the opposite shore.[3]

It would be interesting if the author would give a short technical account of the vertical, geodetic adjustment in his closing discussion, though the readers are presumably familiar with the fact that the corrected triangulation net has, in practice, been brought to the sea-level surface of the earth.

Horizontal Control.—In this project, as well as in many others in the United States, thanks go to the U. S. Coast and Geodetic Survey (USCGS) for its painstaking and precise chains of first- and second-order triangulations that have been partially established during the past decades. Such chains have facilitated both speed and accuracy in the survey control for particular projects.

If the triangulation chain shown in Fig. 5 were entirely a new control survey, it would have been better to avoid a few of the distance angles that appear less than 30°, to strengthen the length computations. Yet, the writer highly commends the synthesis of the partial existing chain and the new connecting chains.

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a January 1961, by James M. Tuttle (Proc. Paper 2702).

2 Chf. Tech. Writer, Palmer and Baker Engrs., Inc., Cons. Engrs., Mobile, Ala.

3 "The 24-Mile Lake Pontchartrain Prestressed Bridge," by Wayne F. Palmer, Proceedings, World Conf. on Prestressed Concrete, San Francisco, Calif., July, 1957, p. 10-1.

The USCGS has done significant work. The writer has done extensive geodetic control work. One was in connection with the development of the Great Northern Port on the Gulf of Pohai, North China and another in the inauguration of the whole water-shed survey of the Yellow River Valley. In both cases, there were neither previous surveys nor nearby existing control work.

The development of the Great Northern Port began in 1929. It involved an extensive triangulation net covering the continental shelf for hydrographic survey, a great port district intended for world commerce throughout the year in a cold climate, and immediate hinterland for projecting railway, highway, and waterway connections to existing transportation systems, meteorogical observations, and wave-dynamometer measurements.

Before the Yellow River Commission was created in 1933, the writer, introduced the control survey, consisting of triangulation nets and precise leveling loops, covering the lower reach of the Yellow River. In 1933, this work was extended to the entire Yellow River system whose main course is more than 2,900 miles with a drainage basin of approximately 205,000 sq miles. By 1943, the survey work had advanced into the upper reach and tributaries.

To begin, it was necessary to undertake control surveys of extensive magnitude, planning and scheduling of work, establishment of standards, organization of survey parties, provision of instruments and equipment, training of crews, unfolding progress in an efficient and operational manner, and institution of office control, computation procedures, and mapping standards.

In the United States, information on federal control surveys can be obtained from either the USCGS or the Map Information Office of the U. S. Geological Survey (USGS). Although their triangulation stations and lines may not be suitable in a particular case for use as base lengths, they will usually supply a starting azimuth and a state plane-coordinate position. These are often lacking in less developed countries. The latter require well trained and resourceful engineers to cope with their multitude of engineering problems under difficult circumstances.

The Heritage and The Future.—Civil engineers have a heritage in surveying and mapping. While it is an art that is as old as any other branch of civil engineering, consideration of geodesy, geodetic astronomy, map projections, adjustments of observation, and astronomical computations shows it to be as scientific and mathematical as any other branch. The introduction of photogrammetry, automatic reduction and plotting, sonic and electronic devices have made it as modern as any other branch of civil engineering.

In 1920, the first-rank institutions required extensive discipline in geodetic engineering. However, both basic and advanced courses in surveying and mapping have been gradually squeezed out of civil engineering curricula and almost eliminated in certain short-sighted institutions. Also, civil engineering projects have grown in magnitude, extent, and complexity on earth; advance plans for mapping other planets and locating space stations must be kept abreast of the development of the space program.

The space program offers an amazing potential for making presently inaccessible measurements. In the spring of 1961, Wallops Island researchers of the National Aeronautics and Space Administration (NASA) hope to pin

down Bermuda's distance from the Virginia coast within 100 ft, reducing the error of 500 ft to 1,000 ft believed to prevail on today's maps.

For better maps, the U. S. Air Force's Cambridge Research Laboratory is working on use of small sounding rockets to establish geodetic bench marks in space. The Air Force plans, in the spring of 1961, to map the distance between 35 locations along the Eastern seaboard, from Cape Cod in Massachusetts down to the Caribbean, by making observations of a rocket-borne flare 1,000 miles above the earth.

The idea is simply to photograph the flare simultaneously from all the ground locations; then, by computations of the varying angles between the earth's surface and the line of sight toward the flare, the distances can be figured between each of the points on the ground. "We feel we can double the accuracy of conventional measurements by this method, going from a precision of one part in 100,000 to one part in 200,000," says Paul Swenson, an Air Force scientist.

While aerial survey and soil-type identification have long been an inter-technological development, the future potentialities of surveying and mapping will unfold as an inter-science discipline.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

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NOVEMBER: 2637(ST11), 2638(ST11), 2639(CO3), 2640(ST11), 2641(SA6), 2642(WW4), 2643(ST11), 2644(HY9), 2645(ST11), 2646(HY9), 2647(WW4), 2648(WW4), 2649(WW4), 2650(ST11), 2651(CO3), 2652(HY9), 2653(HY9), 2654(ST11), 2655(HY9), 2656(HY9), 2657(SA6), 2658(WW4), 2659(WW4)[c], 2660(SA6), 2661(CO3), 2662(CO3), 2663(SA6), 2664(CO3)[c], 2665(HY9)[c], 2666(SA6)[c], 2667(ST11)[c].

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c. Discussion of several papers, grouped by divisions.

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